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DEVELOPMENT OF A LONG DURATION FLOW FACILITY FOR STUDIES OF BLAST-FIRE INTERACTION

Joseph H. Boyes, et al

URS Research Company

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The study reports on the conversion of an underground complex into a Long Duration Flow Facility (LDFF), the calibration of the facility, and a limited test program to study the effect of long duration pressure pulses on extinguishing materials simulated to have been ignited by the coincident thermal pulse (so-called "blast-fire" interaction). The LDFF is composed of a

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High speed photographic cameras and pressure sensing gauges instrument the test room. Three blast-fire interaction tests were conducted and it was found that the blast wave extinguished initial fires, but would not extinguish smoldering fires in upholstered materials such as mattresses. These tests demonstrated the usefulness of the facility.

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FINAL REPORT

Development of a

LONG DURATION FLOW FACILITY FOR

STUDIES OF BLAST-FIRE INTERACTION

by

Joseph H. Boyes
M. Paul Kennedy
C. Wilton

December 1974

for

DEFENSE CIVIL PREPAREDNESS AGENCY
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III

Summary Report

DEVELOPMENT OF A LONG DURATION FLOW
FACILITY FOR STUDIES OF BLAST-FIRE INTERACTION

TYPE OF STUDY

The present study reports on the conversion of the underground complex into a Long Duration Flow Facility (LDFF), the calibration of the facility, and limited test program. The concept of converting a portion of an underground tunnel complex into a long duration blast facility had previously been found feasible through a series of analytical calculations and a small scale model of the proposed facility

PROCEDURE

The LDFF was constructed from an old gun emplacement (adjacent to the URS Shock Tunnel facility at Fort Cronkhite). It is composed of a compression chamber with a volume of approximately 40,000 cubic feet and a test room approximately 12 feet x 15 feet x 9 feet high. The compression chamber is separated from the test room by a mechanical diaphragm. In operation, the compression chamber is filled, using a large air compressor, the diaphragm is opened and the flow vents through the test room, producing a blast wave of up to 5 psi and with a duration of up to 4000 milliseconds.

The test room was especially designed to test the effect of long duration pressure pulses on materials simulated to have been ignited by the initial thermal pulse of a large megaton nuclear weapon. The test chamber may be arrayed as an office, a living room, etc. High speed photographic cameras and pressure sensing gauges instrument the test room.

RESULTS

Three blast-fire interaction tests were conducted to evaluate the facility. In these preliminary tests, it was found that the blast wave extinguished initial fires, except in the case of mattresses. It also appears that the geometry of the openings into the test room play an important part in the degree of fire extinguishment obtained.

Tests in this facility are intended to provide a correlation with earlier similar work conducted using a very short duration blast pulse.

ABSTRACT

The study reports on the conversion of an underground complex into a Long Duration Flow Facility (LDFF), the calibration of the facility, and a limited test program to study the effect of long duration pressure pulses on extinguishing materials simulated to have been ignited by the coincident thermal pulse (so-called "blast fire" interaction). The LDFF is composed of a compression chamber with a volume of approximately 40,000 cubic feet separated by a mechanical diaphragm from a test room approximately twelve feet by fifteen feet by nine feet high.

In operation, the compression chamber is filled; the diaphragm is then opened and the flow vents through the test room producing a flow of up to 5 psi and with a duration of up to 4000 milliseconds (to provide correlation with the long duration pressure pulse of megaton nuclear weapons).

High speed photographic cameras and pressure sensing gauges instrument the test room. Three blast-fire interaction tests were conducted and it was found that the blast wave extinguished initial fires, but would not extinguish smoldering fires in upholstered materials such as mattresses. These tests demonstrated the usefulness of the facility.

FOREWORD

This report presents the results of development of a long duration flow test facility and preliminary tests therein by URS Research Company for the Defense Civil Preparedness Agency. Messrs. Joe Boyes and Paul Kennedy of URS were primarily responsible for the preparation of the test facility and for its operation during the preliminary testing program. Mr. C. Wilton, formerly with URS and now with Scientific Service, Inc., participated in the development of the facility and was responsible under Subcontract No. 7239-74-100 to URS Research Company for compiling the final report, of which he is a coauthor. Dr. Bernard Gabrielson, formerly with URS and now with Scientific Service, Inc., was responsible for all structural analyses.

The aid and assistance of Mr. James Kerr and Dr. Michael Pachuta, Defense Civil Preparedness Agency, is also gratefully acknowledged.

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Section 1
INTRODUCTION AND BACKGROUND

There exists today a scarcity of experimental data on the effects of the long duration flow fields generated by large yield (megaton range) nuclear weapons. Information is needed on such things as the interaction of these flow fields with structures, structural elements, and debris. But of more immediate importance is a need for information on the effects of the flow fields on the rearrangement of kindling fuels and on the extinguishment or enhancement of fires ignited in those fuels.

Some flow processes, for example those involved in filling a simple enclosed space, or those around a simple structure, can be accurately treated analytically in considerable detail. The interaction of blast flows with fires in various substances and geometries is, on the other hand, a vastly more elusive problem, not yet amenable to theoretical analysis. It involves phenomena such as heat transfer, fuel vaporization, mass transfer, and combustion; and the fires can take place in a variety of media (mattresses, cushions, curtains, papers) and in structurally complex arrangements (chairs, beds, etc.). It must, therefore, be studied experimentally. Furthermore, because of the temporal and spatial nature of the phenomena involved, accurate scaling is precluded, and thus experiments must be conducted at full scale.

To carry out such experiments, a facility known as the Long Duration Flow Facility (LDFF) has been developed. In this facility the flows behind the air shock created by the explosion of a weapon in the megaton range can be reproduced under conditions that will allow testing at full scale. The program to develop the facility was conducted in two phases. The first phase, conducted during 1972 under DCPA Contract DAHC20-C-0380, was a feasibility study which included structural investigations of the facility; a limited analytical effort to predict its performance; and the design, construction, and testing of a 1/12 scale model.

The results of this phase are presented in Appendices A and B. The second phase of this program was the construction of the facility and the conduct of a limited calibration test series. It was carried out during the past year and is the main subject of this report. The next section of the report describes the development of the facility, and the last section presents the results of the limited test series.

Section 2
DEVELOPMENT OF THE FACILITY

GENERAL

The basic concept of the facility involved was the utilization of a portion of an existing tunnel complex as a large reservoir of compressed air, and releasing this air in a controlled fashion so that the flow in a testing area resembled the flow behind the shock front of a long duration blast wave interacting with a structure.

The tunnel complex, shown in Fig. 2-1, was originally a coastal defense emplacement consisting of an underground structure with massive, thick concrete walls and roof (the minimum wall thickness in the complex is three feet and the minimum roof thickness is six feet). In addition, the facility has a minimum cover over the roof of approximately 35-ft of earth. Calculations of the strength of this facility relative to its anticipated use were performed during the feasibility phase of the program. A summary of these calculations are presented in Appendix A.

A sketch of the portion of the tunnel complex used as the long duration flow facility is shown in Fig. 2-2. Only relatively minor modifications to the complex were required. These, also shown in Fig. 2-2, included installation of a solid wall (with an access door) at Point A, and a wall with a diaphragm mechanism at Point B. These walls created a compression chamber with a volume of approximately 40,000 ft^3. For the blast fire interaction test program a room was added at Point C. In the remainder of this section, the construction and installation of walls and the diaphragm are described in some detail, and the additional elements required to convert the complex to a flow facility are discussed.

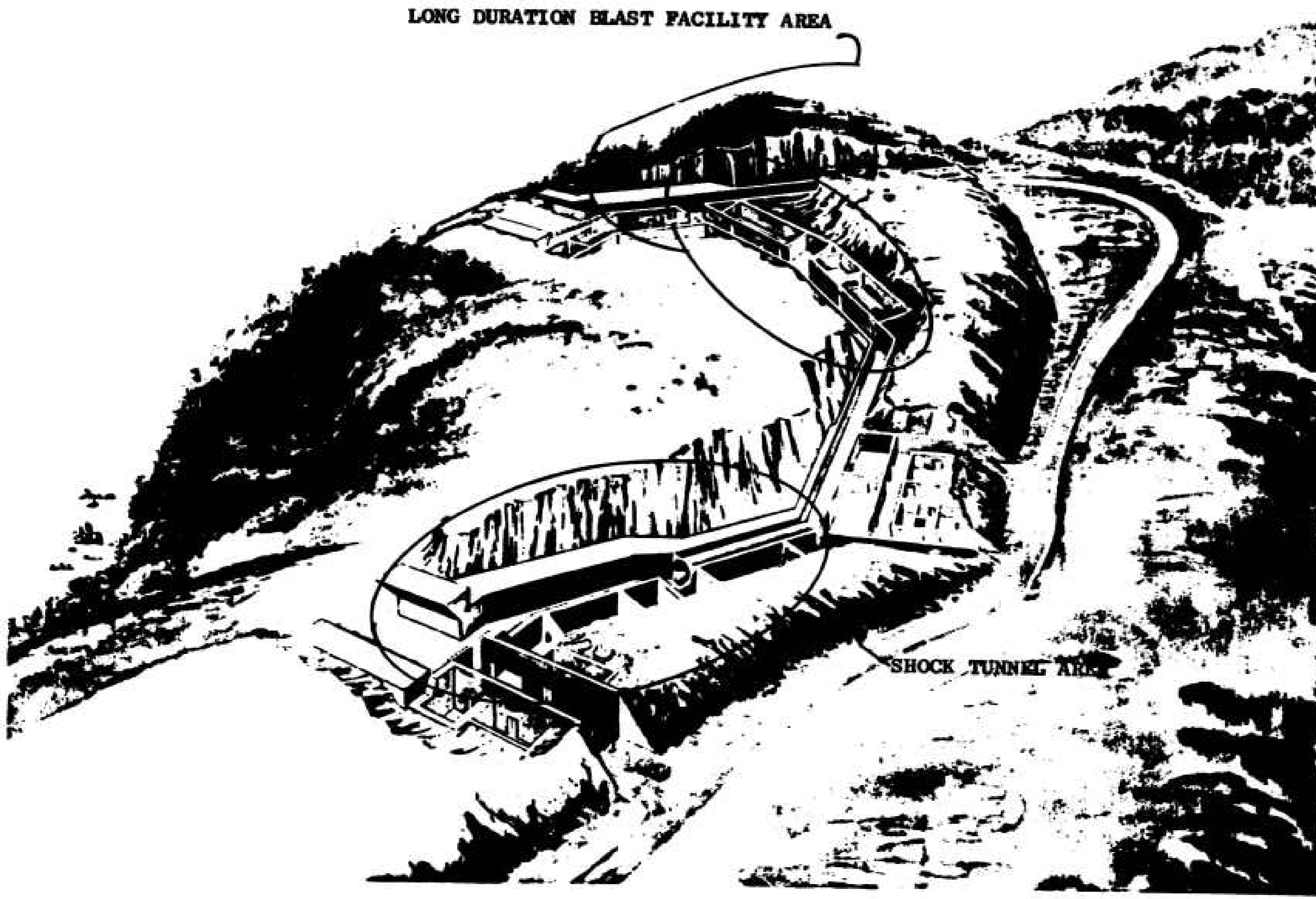


Fig. 2-1. Tunnel Complex.

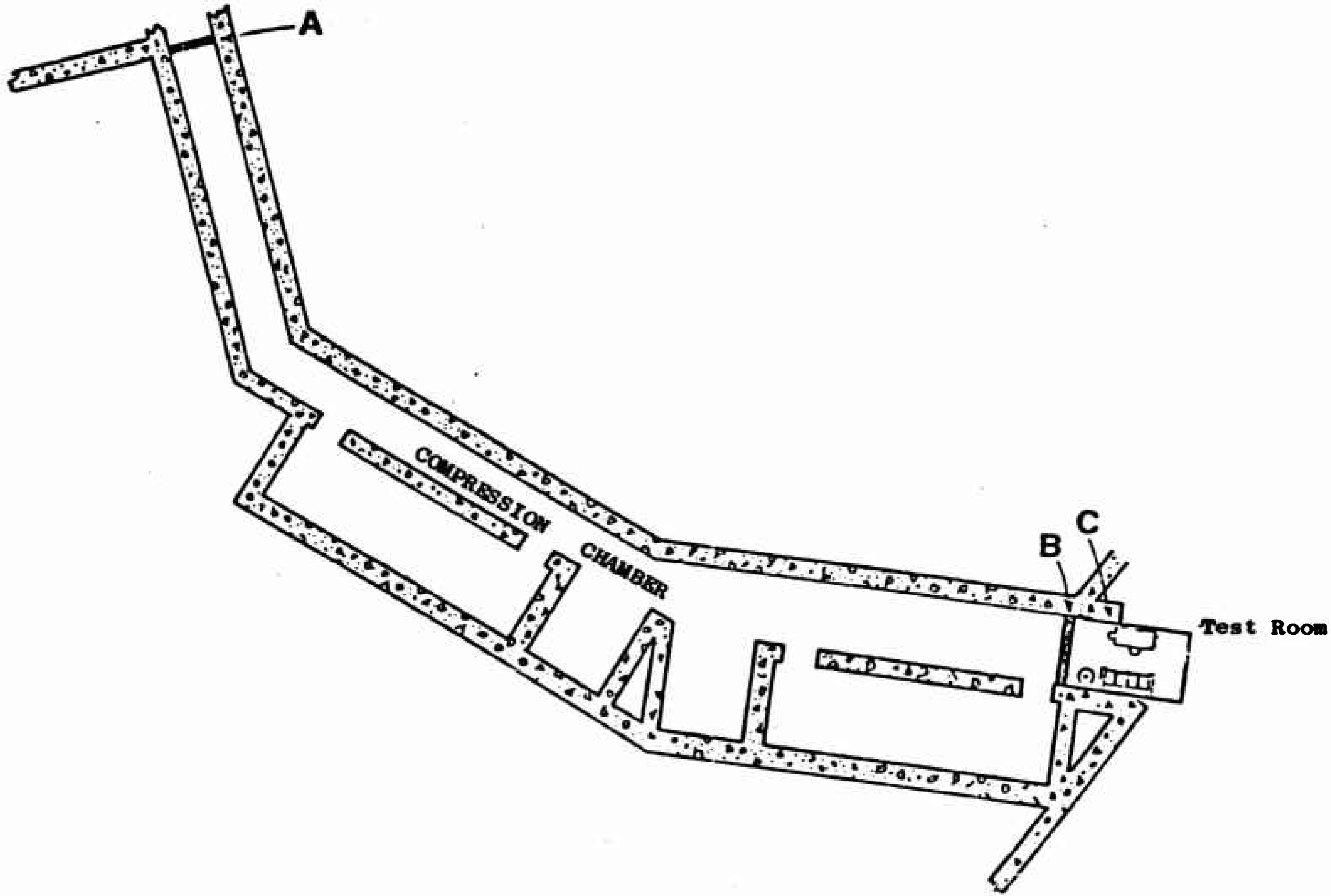


Fig. 2-2. Sketch of Long Duration Flow Tunnel Facility.

COMPRESSION CHAMBER CLOSURE WALLS

For the construction drawings of the walls installed at Points A and B see Fig. 3. The wall at Point A is constructed of laminated plywood and 2 x 4's, with a rigid high strength wood glue being used on all contact surfaces. The basic wall is 12 inches thick; the access door is approximately 3-1/2 inches thick. This wall is supported in the tunnel by laminated plywood support blocks at its top and bottom. To increase the shear capacity of the plywood used in these support blocks, all laminations were oriented 45° to the direction of the load that would be applied by the compressed air in the chamber.

The support blocks were affixed to the sand blasted surface of the floor and ceiling of the tunnel by a rigid epoxy (Concresive #1180). This epoxy has been used extensively in other parts of the Shock Tunnel. Performance tests conducted prior to the installation of the support blocks, indicated that it would provide adequate strength for the blocks.

The gaps around the sides of the wall panels were filled with low density (approximately 2 lbs/cu ft) rigid plastic foam, to provide both an air seal and mechanical strength against rebound.

The access door in the wall at Point A opens inward, that is, into the compression chamber, and is held in place by bolts and the air pressure in the chamber.

Wall B contains the Diaphragm Mechanism and two wood panels similar in construction to those in Wall A. It is held in place by the same type of support blocks and sealed with rigid foam.

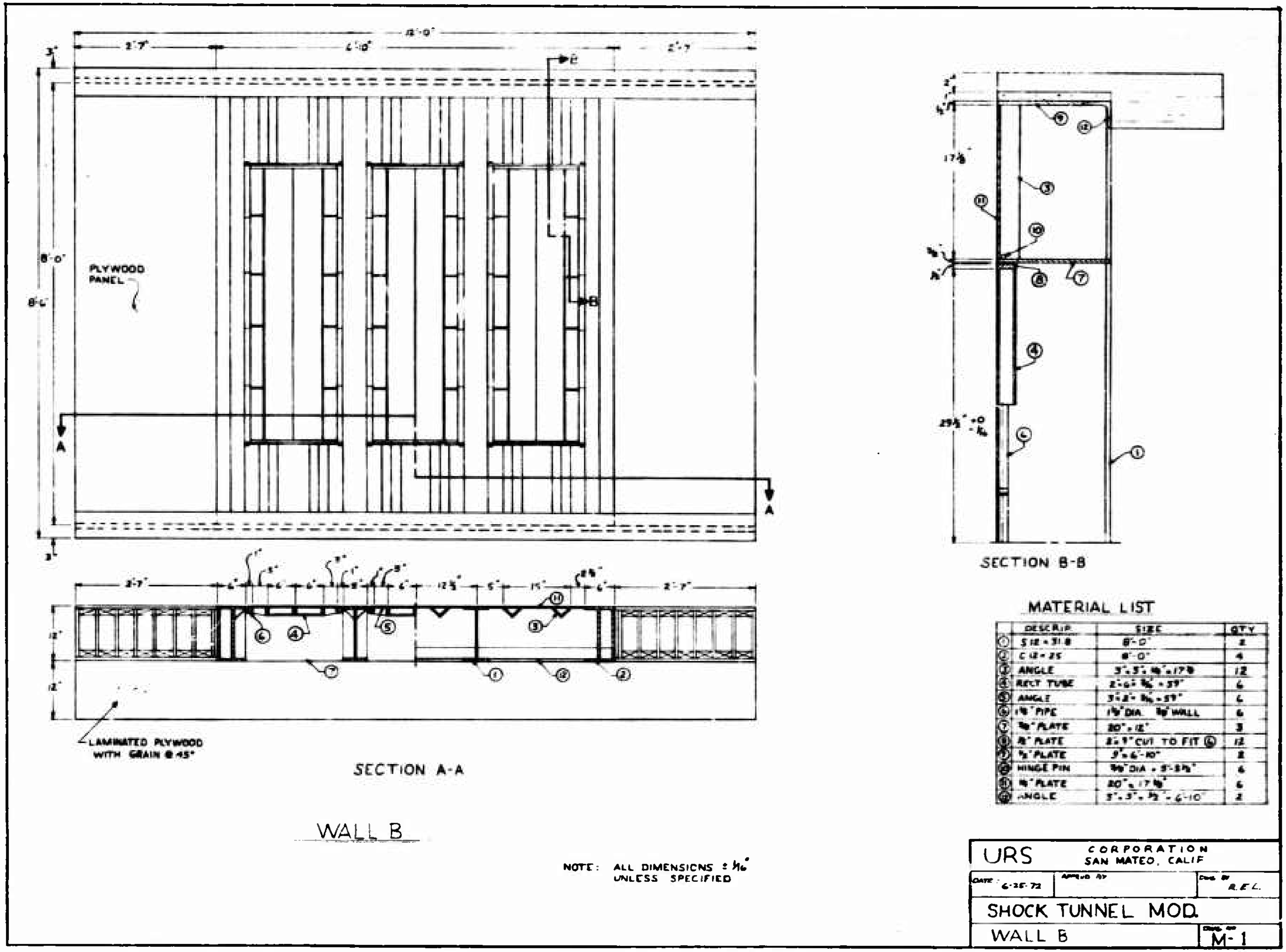


Fig. 2-3A. Construction Drawings of Walls A and B.

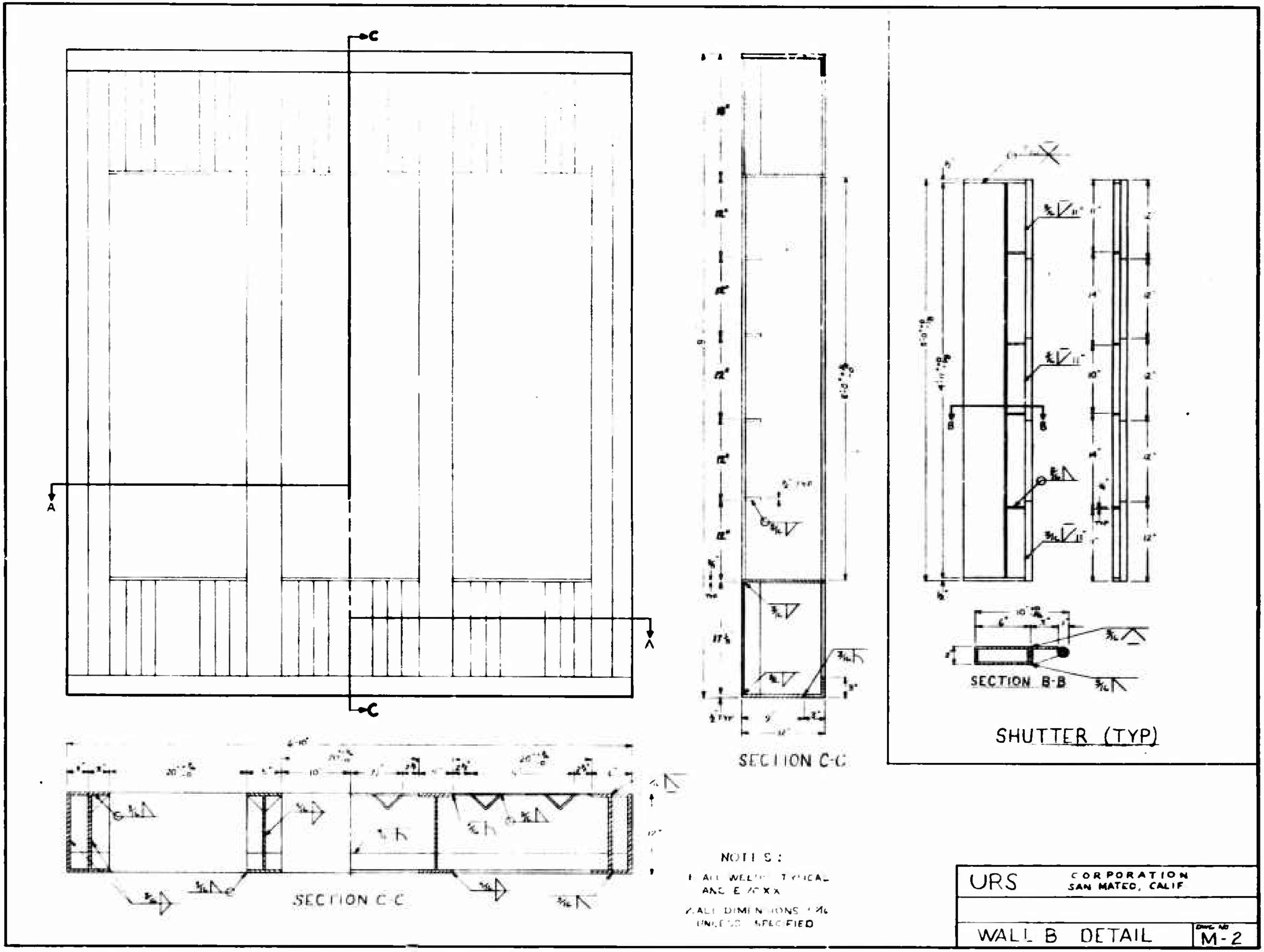


Fig. 2-3B. Construction Drawing of Walls A and B.

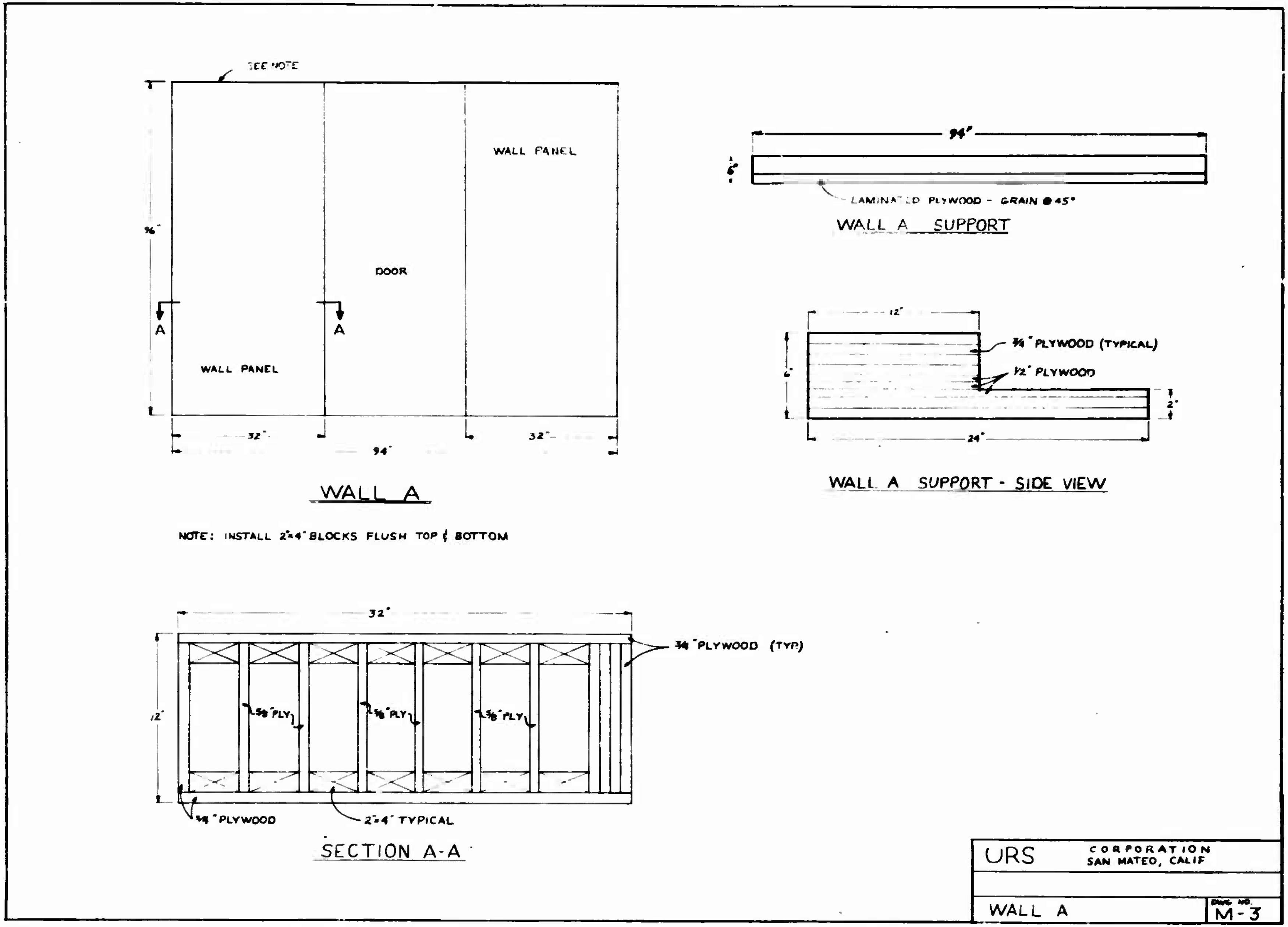


Fig. 2-3C. Construction Drawing of Walls A and B.

DIAPHRAGM MECHANISM

The diaphragm mechanism (construction drawings of which are shown in Fig. 2-3) consists of three sets of vertical shutters. (See Fig. 2-4 for photographs of the system closed as installed in the facility.) With the shutters open there are three open areas, 60 in. high by 17-1/2 in. wide with the total opening area equal to approximately 21 percent of the tunnel cross section. The shutters are kept closed by a support system in which wooden timbers are held against the door by a steel cable as shown in the upper photograph of Fig. 2-5. Cracks around the closed shutters are covered with sheets of .001 in. thick mylar. At the desired reservoir pressure, the steel cable is cut by an explosive-driven guillotine. This releases the doors, which are forced open by the internal pressure in the compression chamber. (See lower photograph of Fig. 2-5.) The mylar diaphragm ruptures very quickly after the doors are released.

MISCELLANEOUS CONVERSION TASKS

Numerous other tasks were accomplished during the conversion of the facility. Among the more important were: cleaning the facility; sealing of leaks; constructing the test room; installing cameras, lights, and instrumentation. The work conducted under each of these tasks is discussed below:

Cleaning the Facility

To prepare for construction and to locate sources of potential leaks as well as to reduce the possibility of dust during operation of the facility, it was necessary to thoroughly clean the compression chamber. This was a formidable task since much of this area had not been used for almost 30 years.

Fig. 2-4. Photographs of Wall at Point B as Installed.

Fig. 2-5. Photographs of Diaphragm Mechanism.

The entire area was swept and washed down and large areas were sand blasted and blown clean with high pressure air. While the primary purpose of the extensive sand blasting was to clean the walls, it also roughened the concrete surface to assure good bonding of the epoxy and rigid foam at wall locations A and B.

Sealing of Leaks

It was noted earlier in the section that epoxy and foam were used to hold and seal the walls. These same materials were also used to seal the major holes in the facility such as the ventilation ducts and floor drains.

It was anticipated that numerous minor leaks would be located in the construction joints and even around the newly installed walls during the preliminary pressurization tests of the facility. The number and extent of these leaks, however, were greatly underestimated. For example significant leaks occured through the foam, through the concrete-plywood joints where there was no adhesion of the epoxy, through the bolt holes in the door, and along the lamination and through surface imperfections in the plywood itself. Sealing of the walls at A and B was accomplished by coating the plywood and foam surfaces with plastic roofing compound, and covering this with 1/8 in. tempered masonite. The masonite serves two purposes: first, it provides mechanical support for the roofing compound, thereby helping to prevent the compound from blowing out through any holes; and second, it provides a smooth surface for seating of the door gasket. Spaces between the shutter mechanism and the plywood were sealed with caulking compound.

After sealing the walls at A and B, it was possible to conduct pressurization tests and detect leaks at construction joints which had been obscured by years of dirt accumulation. These joints were cleaned out by chipping and by blowing the dirt out with high pressure air. The smaller cracks in the floor were filled with roofing compound and the larger cracks with heated roofing tar. Major cracks and joints in the ceiling were sealed with the plastic roofing compound reinforced by strips of .001 in. thick mylar.

It is known that some leaks still remain in the facility. These must be closed to allow operation at high pressures. No difficulties in carrying out the sealing are anticipated; there are numerous epoxys and caulking compounds used in the construction industry for crack sealing which should be adequate.

Construction of Test Room

A test room 12 ft wide, 14 ft-10 in. long and 8-1/2 high was formed at Location C (see Fig. 2-2) downstream from the diaphragm system by installing a wall (with one opening) at the end of the tunnel. Since it was anticipated that further test programs would require many changes in the size of the opening in the back wall, a modular non-failing wall was used. This wall, which had been used for many years for room geometry studies in the shock tunnel, has a steel frame and a series of removable laminated plywood panels. By removing selected panels, a wide variety of window and door opening sizes can be created. A sketch of this wall is shown in Fig. 2-6. This wall is supported in the tunnel by the same type of floor and ceiling blocks used for Walls A and B.

Cameras, Lights, and Instrumentation

Two camera stations and photographic lights were installed in the test room. The locations of stations and lights are noted in Fig. 3-4 in the next section of this report. To monitor the compression chamber pressure, two dial type pressure gauges were installed; one in Wall A, and the other near Wall B. To monitor the blast wave in the test room, two pressure gauges were installed. One in the center of the floor and one in the center of the west wall. These gauges were the quartz piezoelectric type commonly used in the shock tunnel tests. The recording

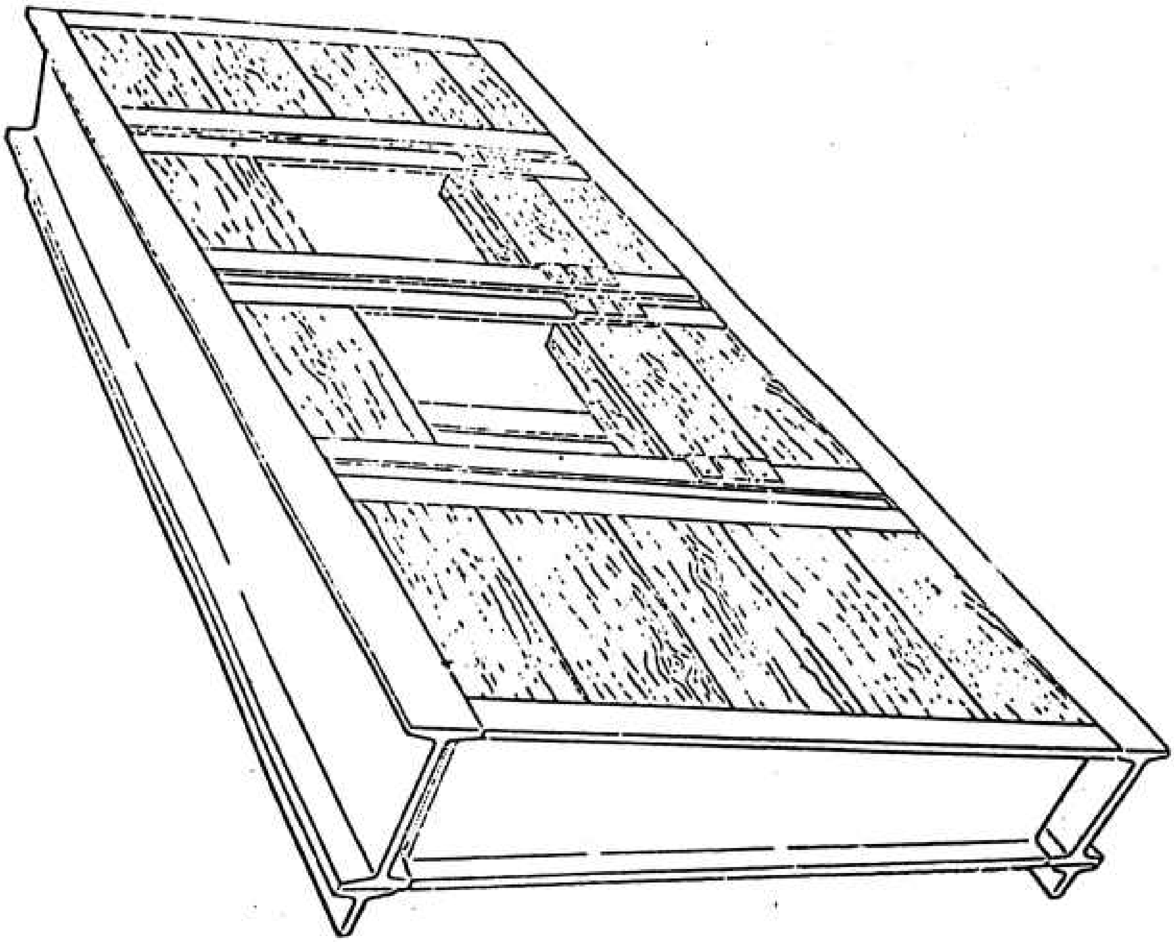


Fig. 2-6. Sketch of Nonfailing Wall.

system used in the shock tunnel itself, which consists of charge amplifiers and a 14-channel tape recorder, was installed in the instrumentation room located directly to the west of the test room.

One additional gauge, a sensitive mechanical leaf switch, was also used to obtain accurate positive phase duration information. This leaf switch was located in the opening of the back wall of the test room.

Section 3
CALIBRATION TEST PROGRAM

GENERAL RESULTS

A limited series of tests were conducted in the facility. These were basically calibration tests, designed to check the operation of the diaphragm mechanism and the instrumentation system, and also to obtain some preliminary performance data. In the last three of these tests, effort was also devoted to obtaining preliminary blast-fire information.

During this test series a variety of experimental arrangements were used to determine the effect of orifice opening size on the peak pressures and positive phase duration obtained in the room. In the front wall two shutter openings (14 percent and 21 percent of the wall area) and in the back wall three doorway openings (12.5 percent, 17 percent and 21 percent of the wall area) were used.

A compression chamber pressure of approximately 2 psi was used for all tests in this series

A summary of data from nine of these tests is presented in Table 3-1. Table 3-1 indicates that positive phase durations ranged from 900 to 1700 msec and the peak overpressure, measured on the side wall of the room, ranged from 0.8 to 1.2 psi. The longest duration, 1700 msec, was obtained with a diaphragm opening of 14 percent and a door opening of 12.5 percent. Typical pressure vs time traces for two of these tests are shown in Figs. 3-1 and 3-2. These tests were conducted using piezoelectric gauges. It is planned to replace piezoelectric gauges - which have serious zero drift over the extended times employed - with strain-type pressure gauges which do not have this limitation.

It is interesting to compare these pulse shapes from those obtained in the 1/12 scale model tests described in Appendix B. Note, for example.

Table 3-1

SUMMARY OF TEST DATA

Number	Diaphragm Opening (%)	Doorway Opening (%)	Peak Overpressure (psi)*	Positive Phase Duration (msec)
1	21	21	0.8	**
2	21	21	0.96	**
3	21	21	0.85	900
4	14	17	0.84	1600
5	14	17	0.84	1500
6	14	12.5	0.91	1700
7	21	12.5	1.2	1200
8	21	21	~0.9	900
9	21	21	~0.9	900

* Measured on the side wall of the test room.

** Flow Gauge data not obtained for these tests.

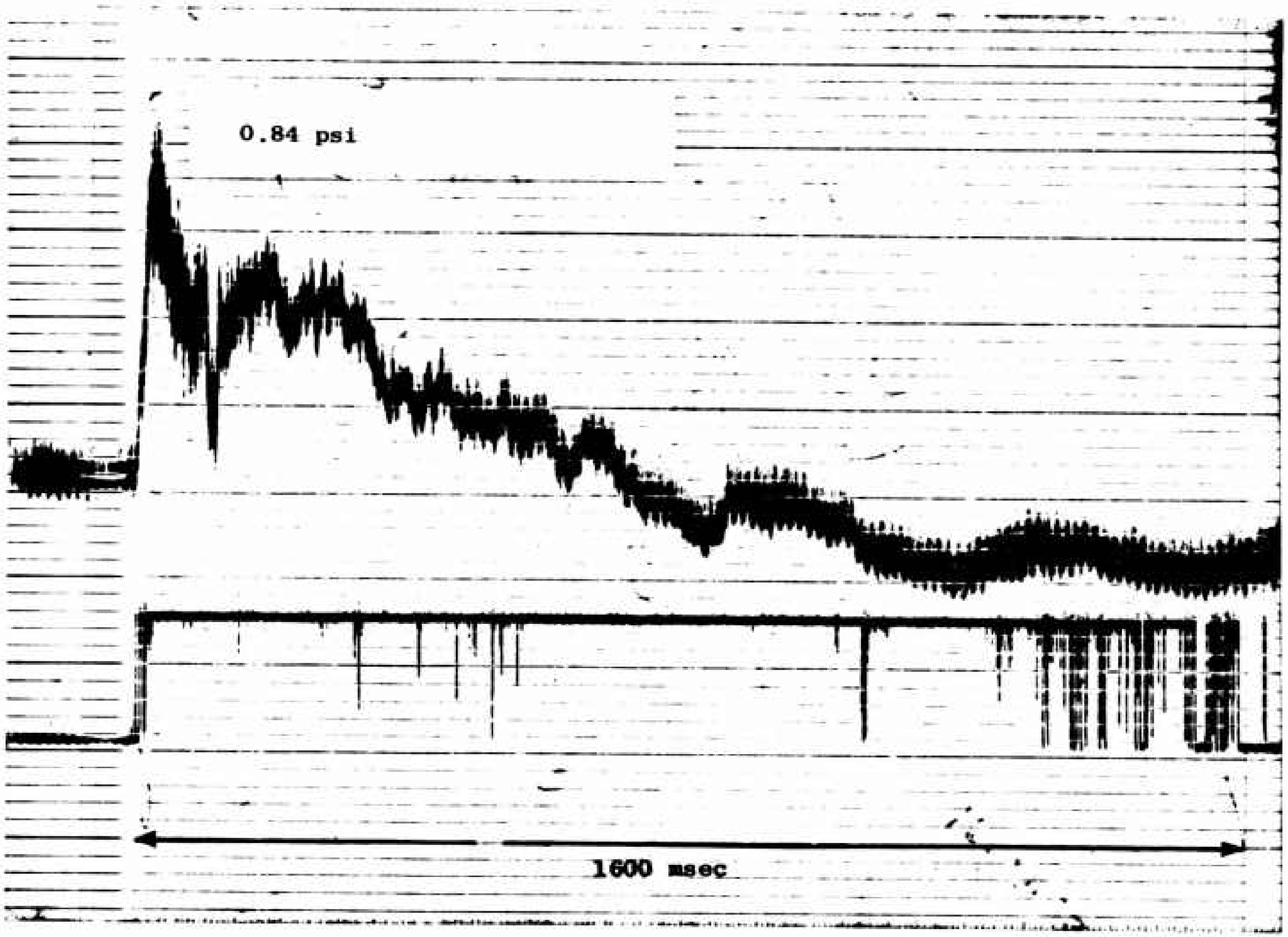


Fig. 3-1. Test 4 Wall Gauge and Flow Gauge Data ~14% Reservoir Opening and ~17% Doorway Opening).

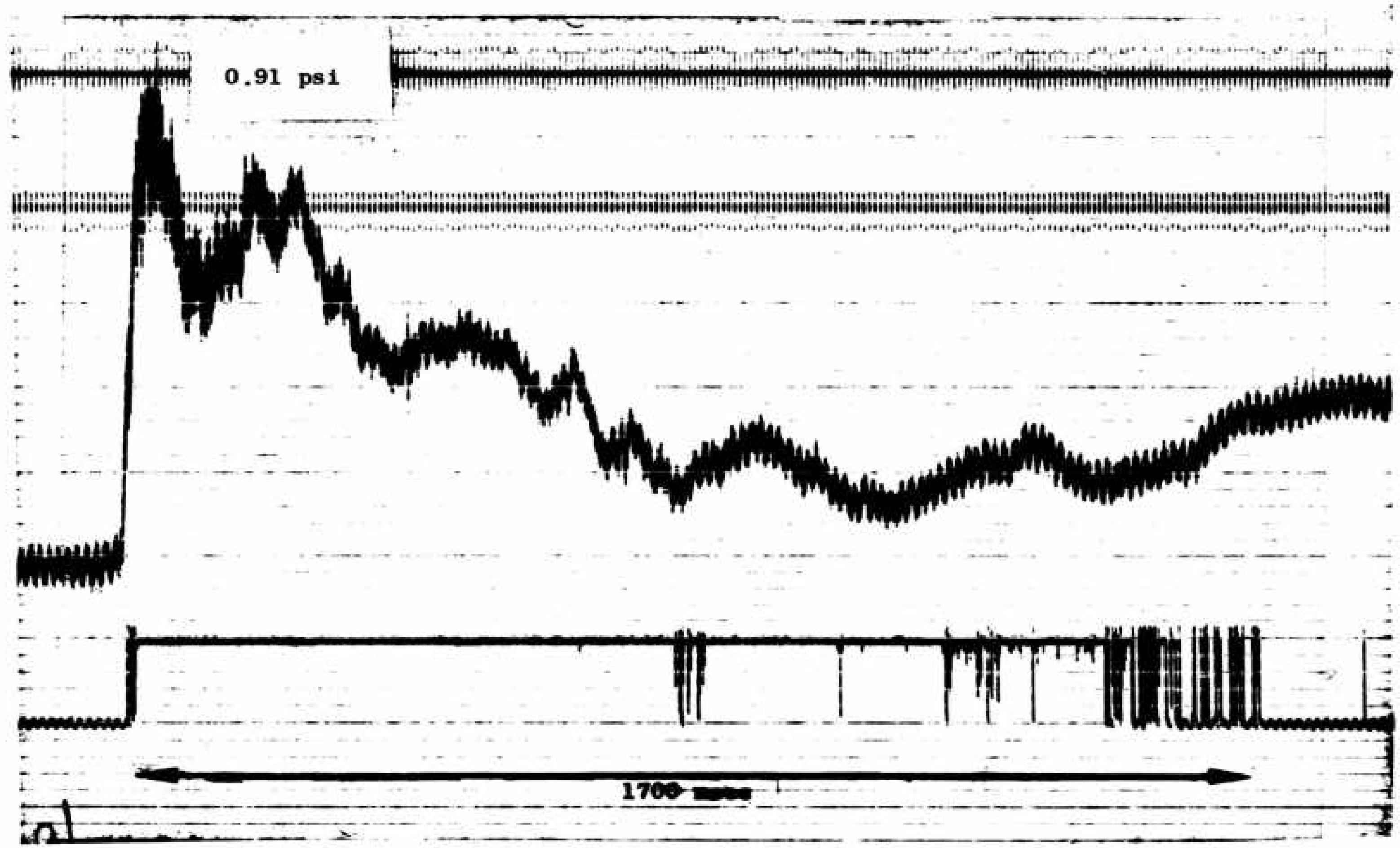


Fig. 3-2. Test 6 Wall Gauge and Flow Gauge Data (~14% Reservoir Opening and ~12% Doorway Opening).

Fig. B-4 a model test at a chamber pressure of 2 psi with orifice opening similar to full scale Test 4. The pulse duration for the model test is approximately 120 msec and the pulse duration for the full scale test is approximately 1600 msec. This full scale duration is somewhat shorter than anticipated (it should be 12 times the model duration) but is clearly of the correct order. Differences probably can be attributed to the different types of diaphragms used, i.e., a single acetate diaphragm in the model, and a series of shutters in the full scale facility.

Four of the tests had photographic coverage using high speed cameras operating at approximately 500 frames per second. Since the shutters are visible in the photographs, data on shutter opening times is also available. These data are presented in Table 3-2. Shutter Number 1 is furthest from the cable cutter, and takes somewhat longer to open fully. (This is probably due to friction of the cable between shutters.) In future testing it may be desirable to use two or even three cutters. Note that in one test, two times are listed, for Shutter Number 2, and these times seem inconsistent with the other three tests. Here, the mylar diaphragm bulged into the opening and did not rupture until 166 msec. The shutters were actually forced open by the bulging mylar rather than by the reservoir air pressure that is applied as soon as the mylar ruptures. Air flow through the shutter did not occur until the mylar ruptured.

Table 3-2

TIME (in msec) TO FULL OPENING
of
SHUTTERS IN RESERVOIR ORIFICE

Test Number	Shutter No. 1 Opening Time (msec)	Shutter No. 2 Opening Time (msec)	Shutter No. 3 Opening Time (msec)
2	173	41	47
4	blocked	73.7	122
8	199	112 (166)*	80.4
9	208	77.4	103

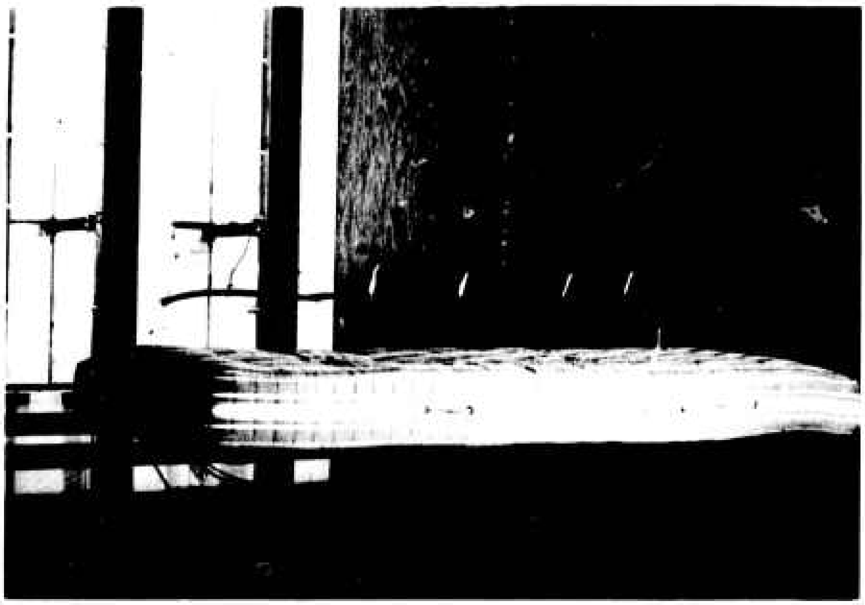
* Time for mylar rupture, see text.

BLAST-FIRE INTERACTION TESTS

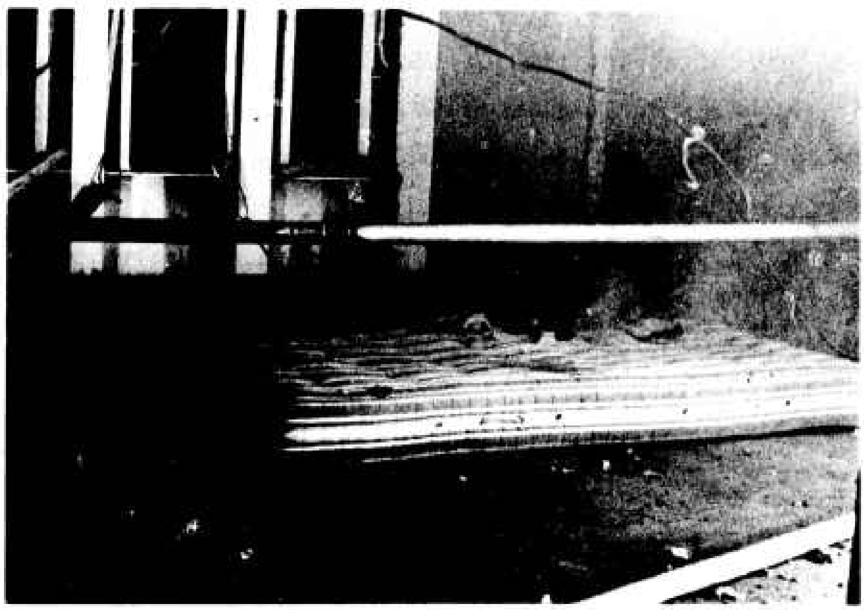
Three tests were conducted with fire sources in the room, two with innerspring mattresses (Tests 4 and 5), and one with the room set up to simulate an office (Text 6). In the mattress tests, one mattress was to be allowed to burn for 30 seconds after ignition and the other 54 seconds after ignition, delay times chosen to correspond approximately to the times between thermal pulse and blast wave arrival in the 1 psi region of 1 MT and 5 MT weapons respectively. The mattresses were placed with their long axes of the mattresses parallel to the wall containing the shutters. They were approximately six feet away from the wall and three feet off the floor. The shutter and the exit doorway openings were both 21 percent of the wall area.

With both mattresses, flames extinguished during the delay time so no flame was present at the time of the shot. However, both mattresses were still smoldering and continued to smolder after the shot. One mattress was flipped over and translated about five feet. The other ended up on the floor below the support and had small pieces of cotton batting torn loose (see pre and posttest photos, Fig. 3-3). One mattress was doused with water after the shot. The other was allowed to smolder, and it re-ignited and burned completely.

In the third blast-fire interaction test (Test 6), an office was simulated by installing a wooden swivel chair and a wooden desk in the test room. Highly flammable materials consisting of a telephone book, two daily newspapers, and a stock of computer output paper were also placed in the room. (See Fig. 3-4 for a room layout.) Most of the paper was neatly distributed on the desk top; part of one newspaper was placed across the chair back. The material was ignited by propane torch, and a delay of 54 seconds was used between ignition and blast arrival.



A - Pretest



B - Posttest

Fig. 3-3. Pre and Posttest Photographs of Test 4.

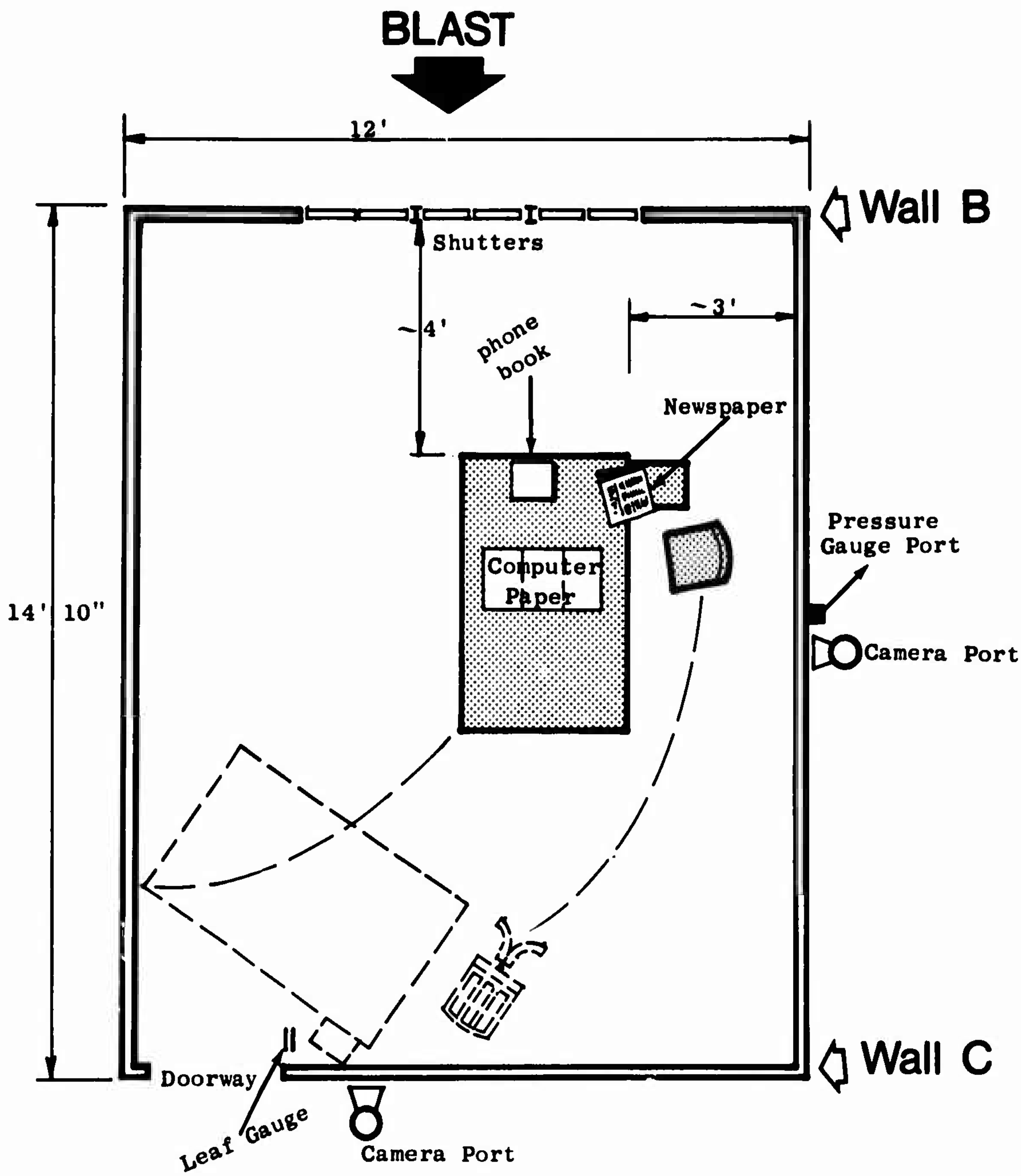


Fig. 3-4. Floor Plan of Office Showing Pre and Posttest Positions of Furniture (Test 6).

Fire was evident at time of blast arrival. The chair translated about eight feet and tipped on its front side (see Fig. 3-5A and B). The desk translated about eight feet and rotated about 135° clockwise. Two drawers were out of the desk. One of these was twelve feet outside of the room. Translation of the desk was terminated by contact with the wall and substantial damage was done to the desk by the impact. The major portion of the telephone book remained intact (see Fig. 3-6A) and in the room. Most of the loose paper blew out of the room through the door (see Fig. 3-6B). There was no reignition or smoldering after two hours. It is assumed that all ignition sources were extinguished and no rekindling would occur. (Note that the peak overpressure in the room was 0.84 psi, and the flow duration was approximately 1600 msec.)

Previous testing done in the shock tunnel* employed shock waves whose durations were on the order of 100 msec, i.e., much shorter duration than can be obtained in the new facility. Although the shock tunnel test condition differed somewhat from those in the new facility, three of these tests are of interest for a preliminary comparison. A shock tunnel test with an office configuration at an overpressure level of 1.1 psi and a duration of 90 msec did not extinguish the flame in the papers. This failure to extinguish is primarily attributable to the short duration of the shock front but may be related, as a second order effect, to the size of the window opening (51 percent in this case). Certainly it is recognized that the opening size is an important consideration with a long duration front, although the exact mechanism is yet to be resolved. Two shots at about the same overpressure level were done with a living room configuration. In both of these shots the window opening was 14.4 percent, approximately the same as that used in the facility. In both cases flames were not extinguished, and most of the debris stayed in the room.

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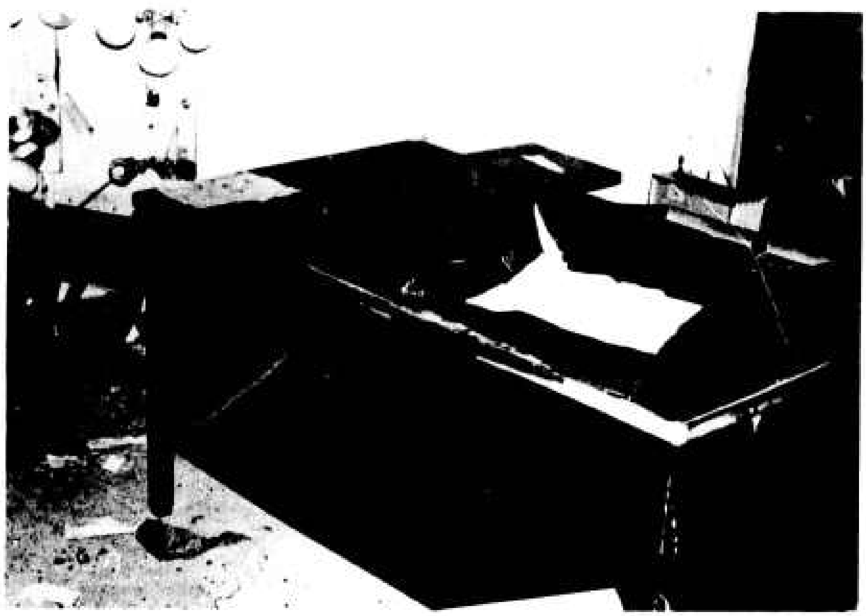
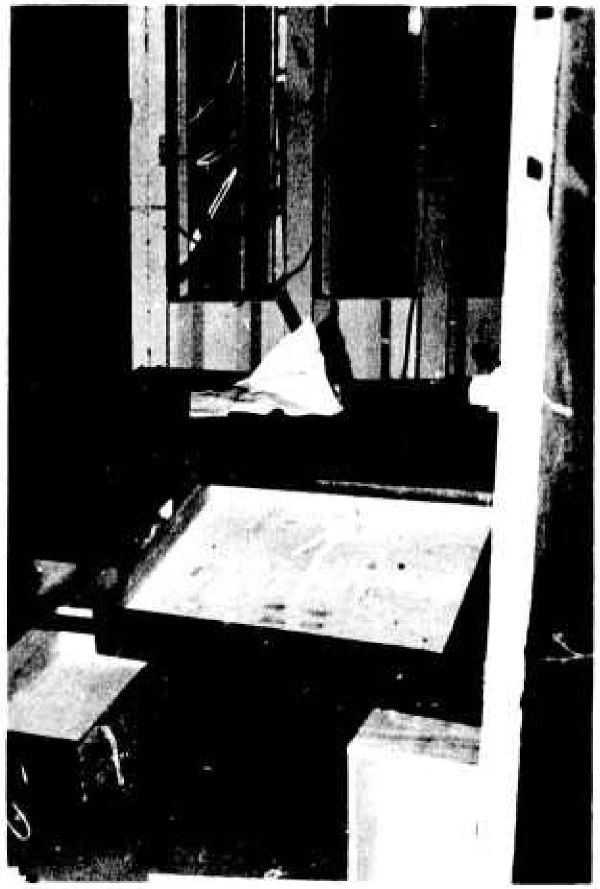
* Goodale, Thomas, Effects of Air Blast on Urban Fires, URS 7009-4, URS Research Company, December 1970.

A



B

Fig. 3-5. Posttest Photographs, Test No. 6.



A

B

Fig. 3-6. Posttest Photographs, Test 6

From these preliminary comparisons, it does appear that there may be significant differences in blast-fire interactions and kindling fuel re-arrangement due to longer flow durations.

Appendix A
STRUCTURAL CONSIDERATIONS

One aspect of the feasibility phase of the program was to examine the tunnel structure to determine safe operation criteria and, also, to determine if any modifications of the existing structure would be required.

The part of the tunnel complex of interest is shown in plan view in Fig. A-1, along with soil contours. Figures A-2, A-4 and A-5 are from construction drawings of the facility, and show wall details in the general vicinity of the bend in the tunnel complex near the center of Fig. A-1. Note that the region characteristically contains a tunnel with various rooms opening off it. An elevation through the area is shown on Fig. A-5; the tunnel is on the left, and one of the rooms is on the right.

Initial analysis allowed for a safety factor of four to catastrophic failure. This was done so that if the design of hardware was also maintained at a safety factor of four, after some experience and proof testing, the operating level could be increased as much as 50%, while still maintaining a safety factor ≥ 2.67.

Assumptions concerning concrete properties, soil properties, and construction details were based on three sources of information, namely:

- The original drawings
- Observations of the behavior of the "Shock Tunnel" portion of the facility
- Original assumptions from the "Shock Tunnel" analysis modified by four years of observation and use.

The analysis that follows is divided into three parts:

- Gross behavior of the facility

- Behavior of weaker portions of the facility
- Consideration of miscellaneous details.

GROSS BEHAVIOR

The first cut at a gross-catastrophic failure was based on:

- No resistance of the concrete (i.e., the structure disassembles at the construction joints)
- Soil properties as follows:

$$\gamma = 110 \text{ pcf}$$
$$\phi = 15^{o}$$
$$p_s = 10{,}000 \text{ psf}$$
$$p_{s(ult)} = 20{,}000 \text{ psf}$$

use

$$K = \tan^2 \left(45 \pm \frac{\phi}{2}\right)$$

The gross elements of the problem are shown in Fig. A-6

Details of the calculations are shown on calculation sheet No. 1,* but the conclusion is that the internal pressure required to lift the concrete and soil above the facility (assuming no concrete strength, and no friction along the soil failure plane) would be 40 psi.

WEAKER ELEMENTS

The weakest element of the facility appears to be the 3-ft thick outer walls (Wall G in Figs. A-2, A-3, and A-4) and the 3-1/2-ft thick wall with cable chase (Wall R in Figs. A-2, A-3, and A-4). These are the two outer walls in Figs. A-5 and A-6.

These walls are very similar in construction to the weakest interior partition in the shock tunnel portion of the complex. This wall has been subjected to peak reflected overpressures of 20 psi without any detectable

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* Calculation sheets are attached at the end of this Appendix.

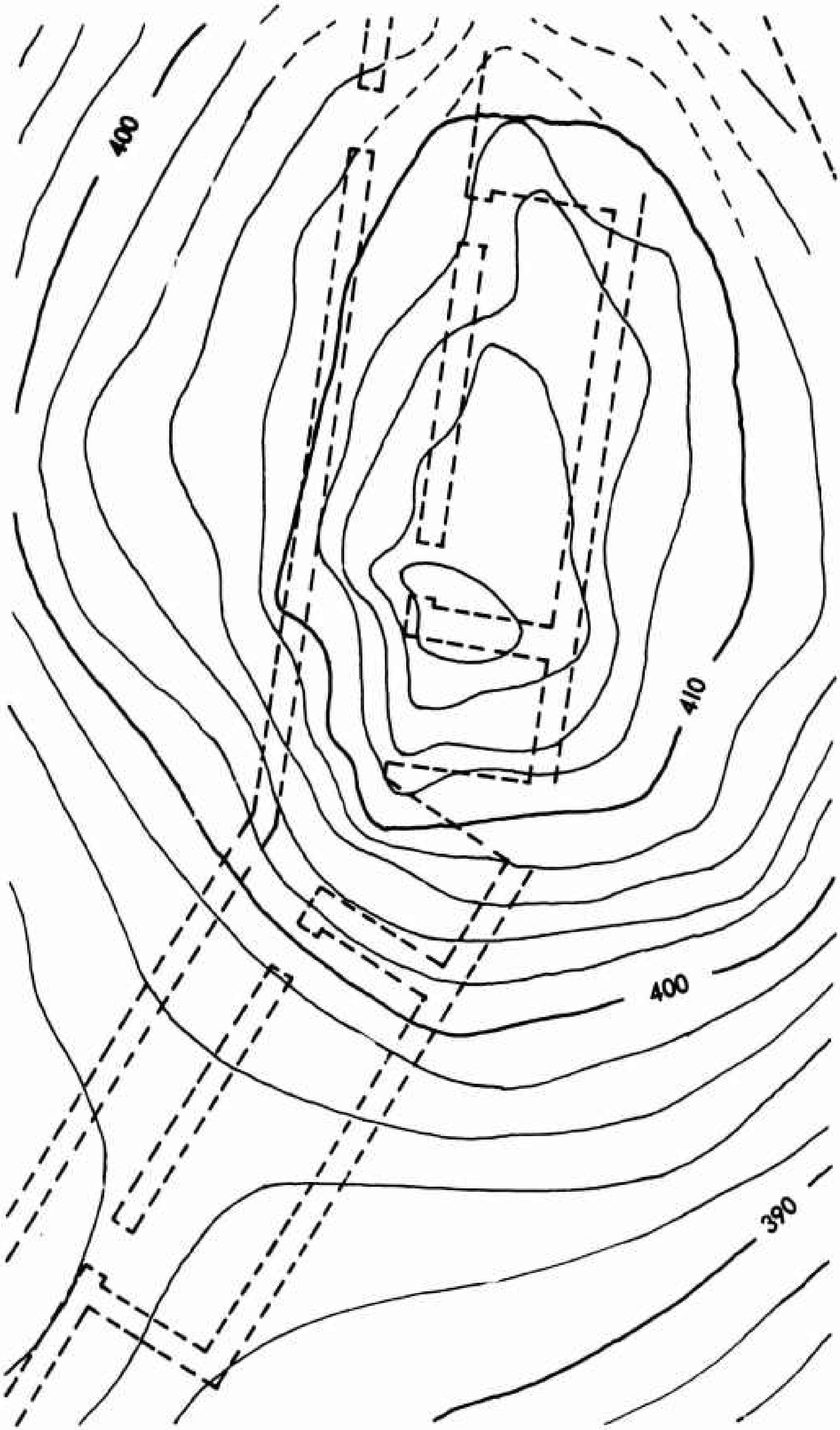


Fig. A-1. Facility Plan View With Contours.

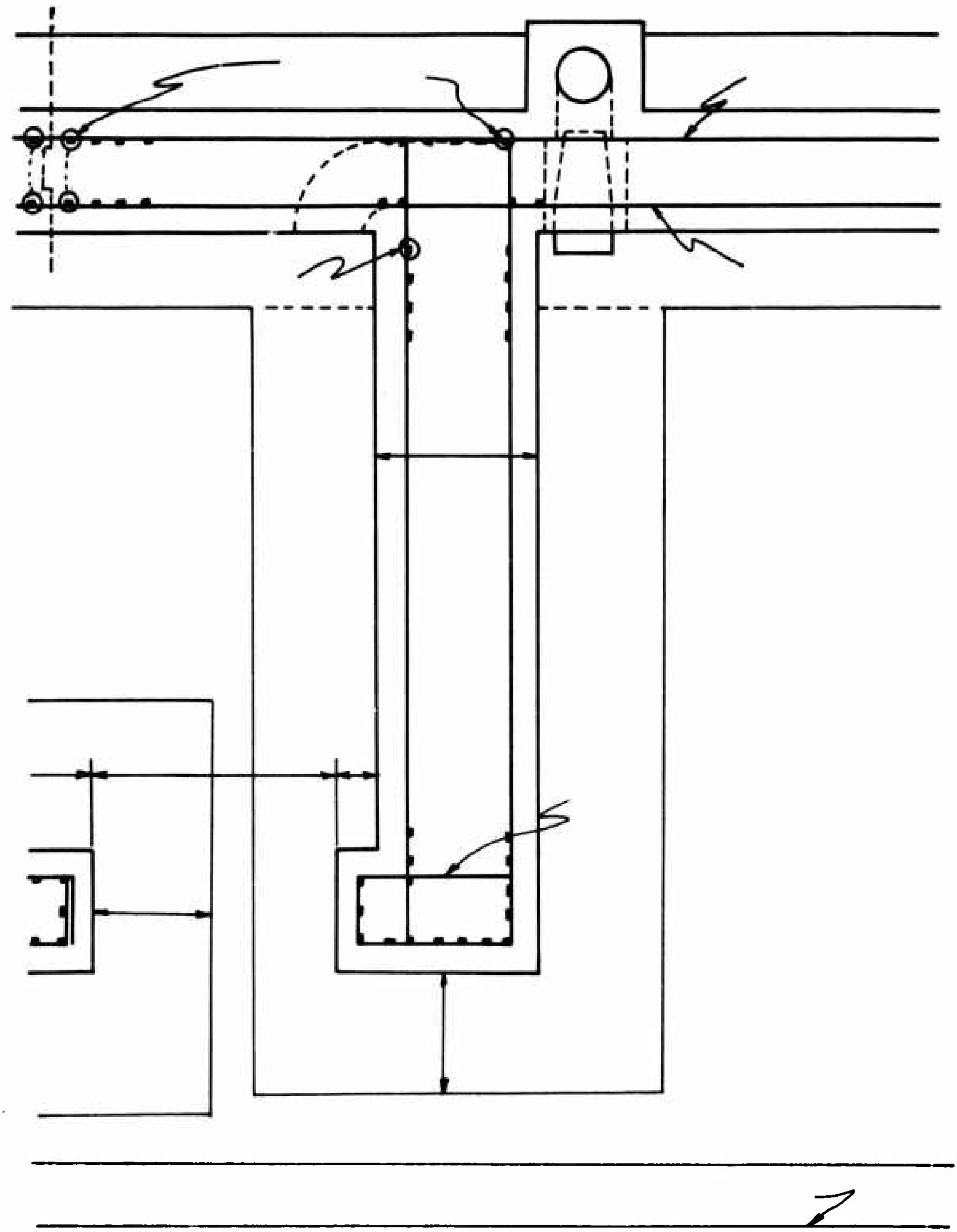


Fig. A-2. Typical Wall Cross Section (Plan View)

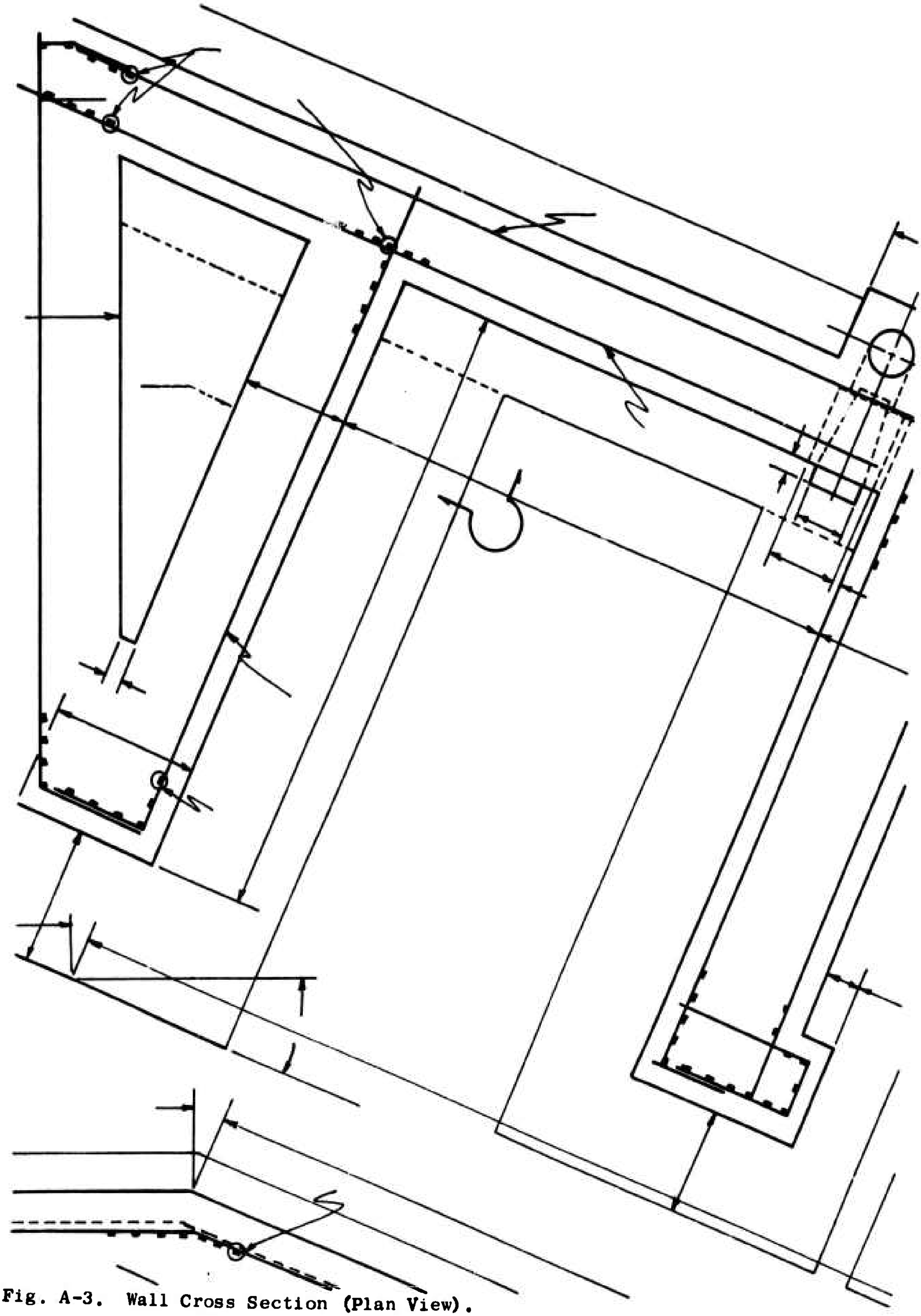


Fig. A-3. Wall Cross Section (Plan View).

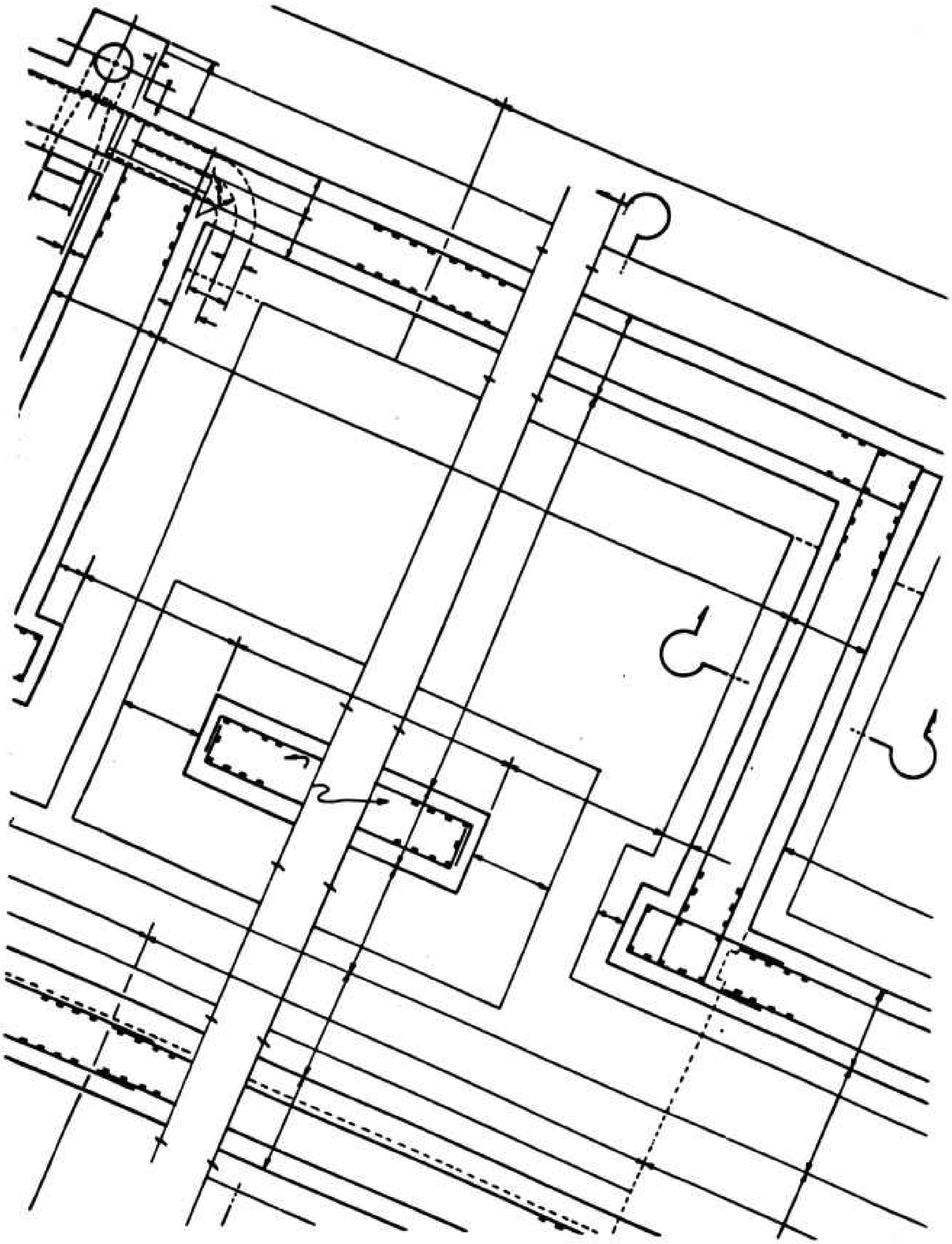


Fig. A-4. Cross Section of Walls (Plan View)

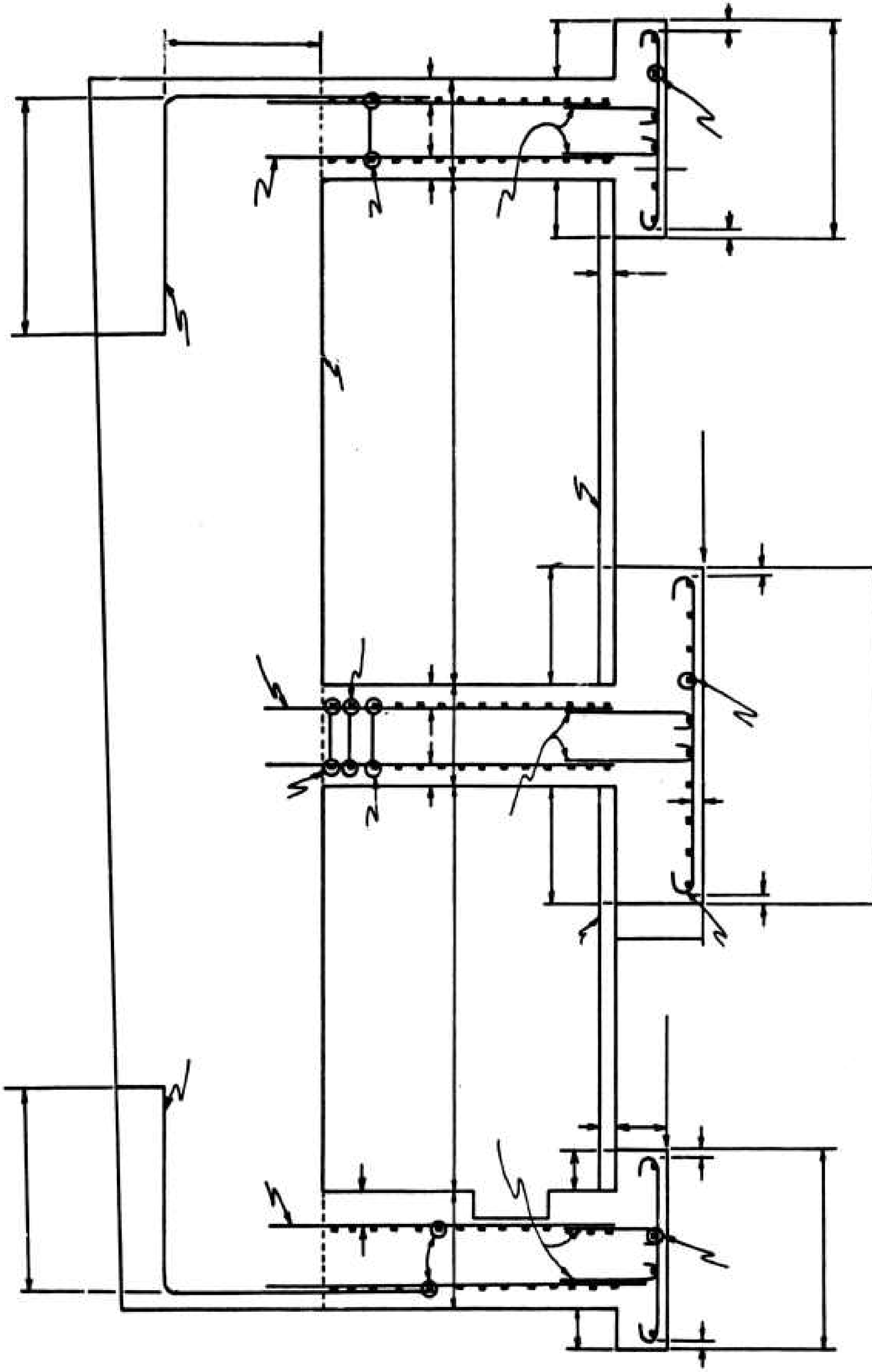


Fig. A-5. Cross Section Through Complex.

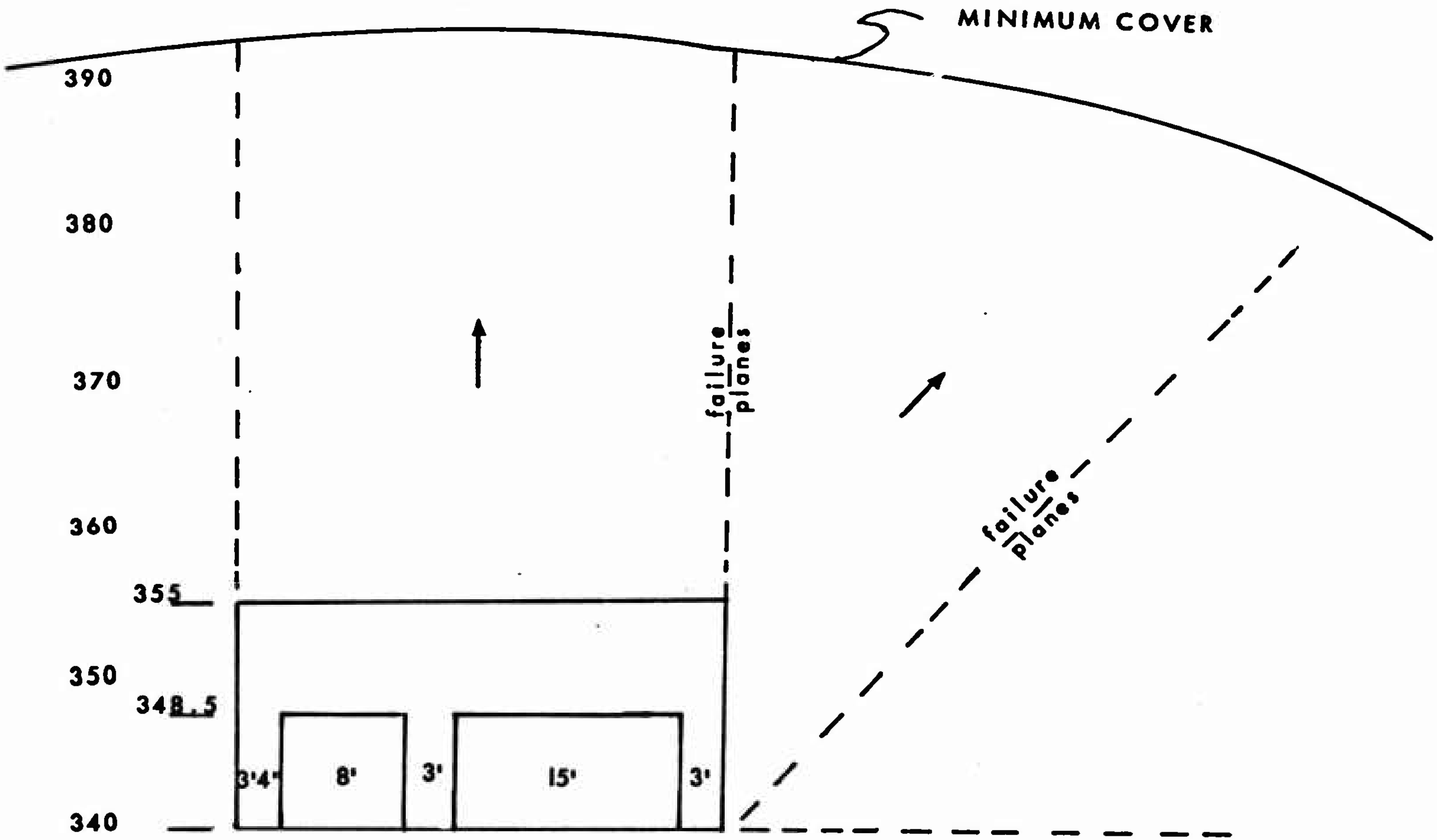


Fig. A-6. Cross Section of Facility With Assumed Soil Failure Lines.

deterioration, i.e., it has remained entirely elastic. Allowing a dynamics load factor (DLF) of two, it appears that the wall is safe for at least 10 psi with a safety factor of four without further analysis. For safety, however, another analysis was made using the following assumptions.

1. No vertical load (very conservative)
2. The wall acts like a one-way slab
3. fc' = 3,000 psi
   fy = 36,000 psi
4. The wall acts as a fixed-fixed beam in the ultimate. (There is sufficient anchorage in the footing and roof, and it is fairly ductile because of the low reinforcement percentages.)

Details of the calculations are shown on calculation sheets 2, 3, 4, and 5, but the conclusions are that:

1. In both flexure and shear (not considering any earth backing), failure requires more than 40 psi internal pressure
2. Bond and cold joints are adequate based on earlier shock tunnel calculations
3. Active soil pressure and the dowel action of the re-bar at the joints enhance the safety factor.
4. Passive soil pressure alone would withstand pressures greater than 40 psi.

SUMMARY

The preceding shows that:

1. The weight of the soil and concrete above the facility is enough to withstand 40 psi internal pressure (assuming no concrete strength).

2. The internal pressure is essentially static, that is , it takes hours to build up to the operating pressure and the release is relatively slow (seconds). Hence, an operating pressure of 10 psi provides a safety factor of four and an operating pressure of 15 psi provides a safety factor of 2.67.

3. The above is extremely conservative since it ignores the strength of the concrete. In addition, the assumptions concerning the roof-soil are very conservative.

For the Roof to Move

Soil Wt. = (390-355)(0.110)(32.3') = 124 kips/ft
Conc. Wt. = (355-348.5)(0.150)(32.3' = 31 kips/ft
155 kips/ft

Lifting Force

Min. = p_s 23 ft^2/ft

Max. = p_s 32.3 ft^2/ft ← full leak @ joint

Lift > Mass → failure

Min. $p_s \geq 4.80$ kips/ft^2 (33 psi)

Max. $p_s > 6.75$ kips/ft^2 (47 psi)

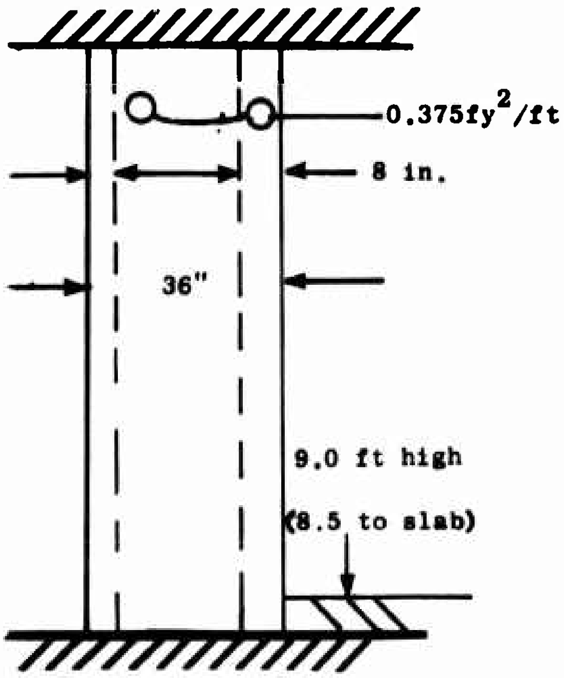
If we assume a pressure gradient at the joint p_s = 5.76K/ft^2 or p_s = 40 psi

ult. p_s = 40 psi
for F.S. - 4
Service p_s = 10 psi

Note concrete is assumed to have no strength.

Calculation Sheet No. 1

Check Wall (Flexural Strength)



$d = 28$ in.

$$a = \frac{As\ fy}{6085\ fc'}$$

$$a = \frac{0.375(36,000)}{.85(12)(3,000)}$$

$a = 4.42$ in.

$\bar{j}d = d - a/2$

$jd = 25.79$ in.

$\bar{M} = As\ fy\ jd$

$$\bar{M} = 0.375(36)\frac{25.74}{12}$$

$\bar{M} = 29.1$ K-ft

$Mu = 26.1$K-ft/ft

Failure if $p\ell^2 \geq 2\ Mu$

$$p_{su} \geq \frac{16}{2}\ Mu$$

Calculation Sheet No. 2

$$p_{su} = \frac{16(26.1)}{81}$$

$$= 5.17\ K/ft^2$$

$$\text{or } p_{su} \geqslant \underline{\underline{36\ psi}}$$

$$\bar{p}_s \geqslant 40 \text{ psi @ ultimate}$$

Flexural Failure therefore
p_s = 10 psi o.k. with F.S = 4

Check Shear

Max. Reaction $\bar{V}$ = 4.75(5.76)

= 27.3 kips

$$\bar{v} = \frac{27,300}{12(28)}$$

$\bar{v}$ = 81.4 psi < 100 psi ∴ safe

Bond @ Cold Joints

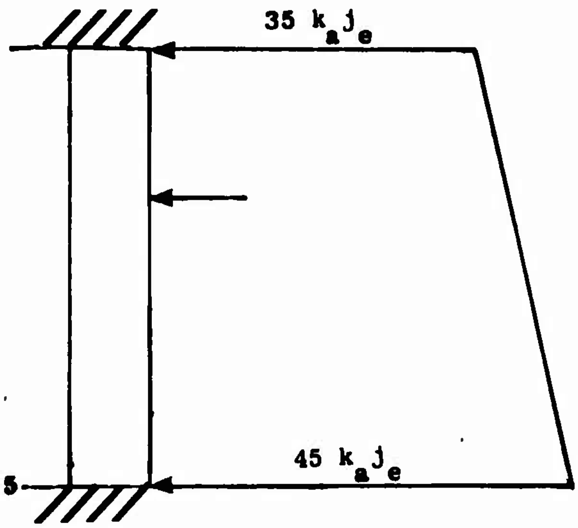
From our previous analyses 90 < $\bar{v}_c$ < 270
hence the joint bond is still intact.

Calculation Sheet No. 3

Additional Factors

1) The re-bar acts as dowels at joint hence create a reserve shear capacity

2) If there is active soil pressure it will also enhance the safety factor.



$$K_a = \tan^2(45-\frac{\phi}{2})$$

$$K_a = 0.33$$

Active pressure (ave.)

$$p_a = \frac{(35+44)}{2}(0.33)110)$$

$$= 1300 \text{ psi}$$

$$\underline{\underline{p_a = 9.0 \text{ psi}}}$$

Which is almost as much as the operational pressure

Calculation Sheet No. 4

3) Passive Reaction of wall if failure occurred in the concrete and only soil remained

$$K_p = \tan^2(45+\ )$$

$$K_p = 3.0$$

$$P_p = \frac{(35+44)}{2}(3.0)110$$

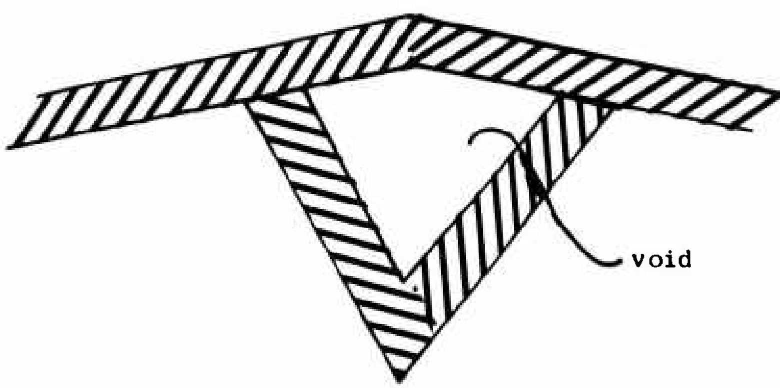
$$P_p = 13{,}000 \text{ psi}$$

P_r = 90 psi hence walls cannot punch out

$\therefore P_{ult.} \geqslant 40$ by four based on a roof failure

Miscellaneous Details

One region of the tunnel has a couple of triangular voids inside of



3' thick partitions with re-bar on one side only such that

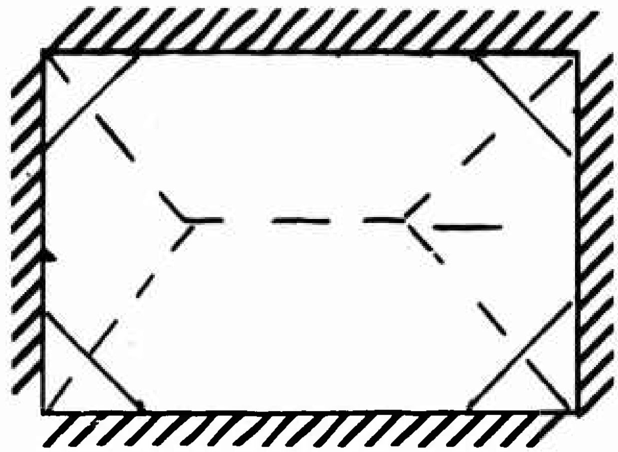
d = 28 in for negative moment

and d = 8 in for positive moment

Calculation Sheet No. 5

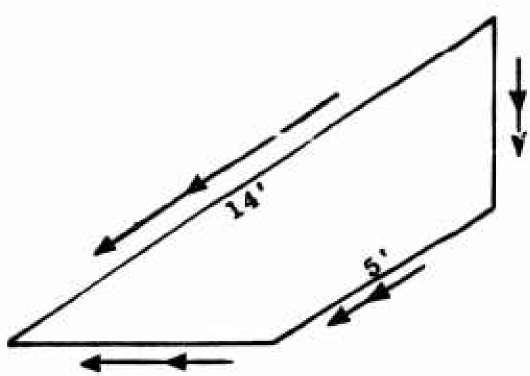
$\therefore$ From p2[6], $\bar{M}^{(-)} = 29.1$ k-ft/ft

$\bar{M}^{(+)} = 6.51$ k-ft



Treat as a 9' x 14' fixed 2-way slab

First cut @ 45° = (if in ballpark let slide)



Resistance Total

$$\bar{M}_r = 14x29.1 = 407$$

$$5x\ 6.5 = 33$$

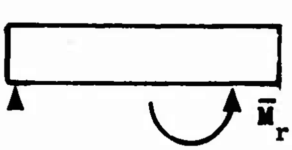
$$\frac{9x\ 6.5}{2} = \frac{41}{481}$$

$$\bar{M}_\ell = \bar{p}_s\ \frac{9x(14+5)}{2}\frac{x9}{2}$$

$$\bar{M}_\ell = \bar{p}_s\ 96$$

$M_\ell > M_r$ failure

$$p_s = \frac{481}{96} = 5.01 \text{ kst}$$



$\bar{M}_r$

$$\underline{\underline{p_s = 3.48 \text{ psi}}}$$

Calculation Sheet No. 6

This detail suggests possible cracking of an interior portion at 35± psi which, of course, would bleed pressure into the void and relieve the problem. Further analysis would probably push this to 40 psi also, but this is felt to be wasted time at this point.

Calculation Sheet No. 7

Appendix B

SCALE MODEL TESTS

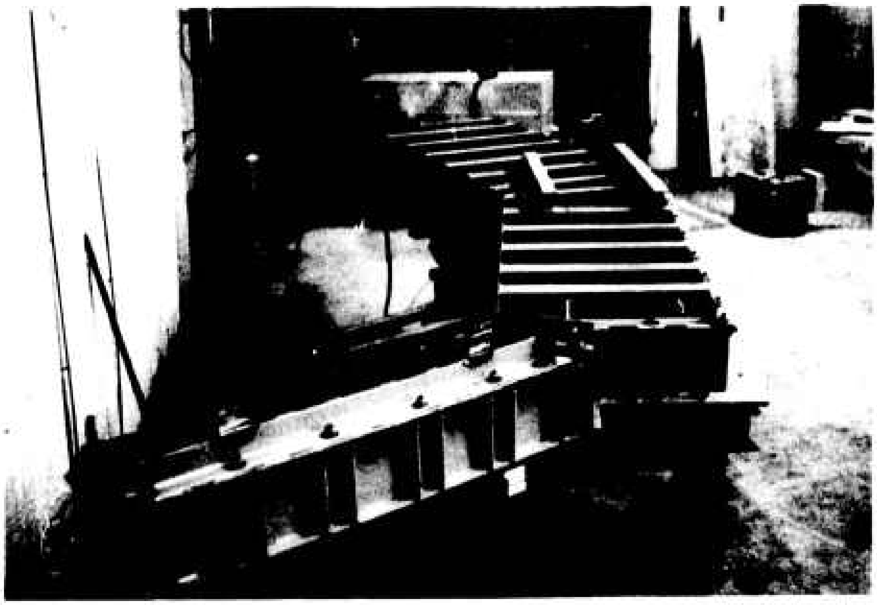
DESCRIPTION OF THE MODEL

To demonstrate the feasibility of the concept and to obtain data for the design of the diaphragm system, a 1/12 scale model of the facility was designed and constructed.

The model, an accurate scale model of the tunnel area to be used for the full scale facility, is approximately 20 ft long, of welded steel construction and weighs about 3000 lbs. Photographs of this model are shown in Figs. B-1 and B-2. The exterior walls were fabricated of 1/2 in. thick steel plate with numerous stiffeners, and the interior portions were fabricated of 1/4 in. and 3/8 in. thick steel plate. Continuous welding was used on all interior joints to make it airtight and structural analysis dictated the welds used on all exterior joints and stiffeners. The structural analysis and design considerations used for this model are presented at the end of this Appendix.

The roof of the model is sealed with rubber gaskets and held in place by bolts. This allows the roof to be removed allowing alterations to the model to be studied experimentally prior to any full scale modifications to the LDFF itself. It is anticipated that this model will be of great value in much of the future work, since it permits scale model tests of experimental configurations prior to full—scale tests. This will help to determine the effect of geometric changes such as changing the window, door or room size on the overpressure pulse shape, pressure level and duration, prior to conducting full scale tests.

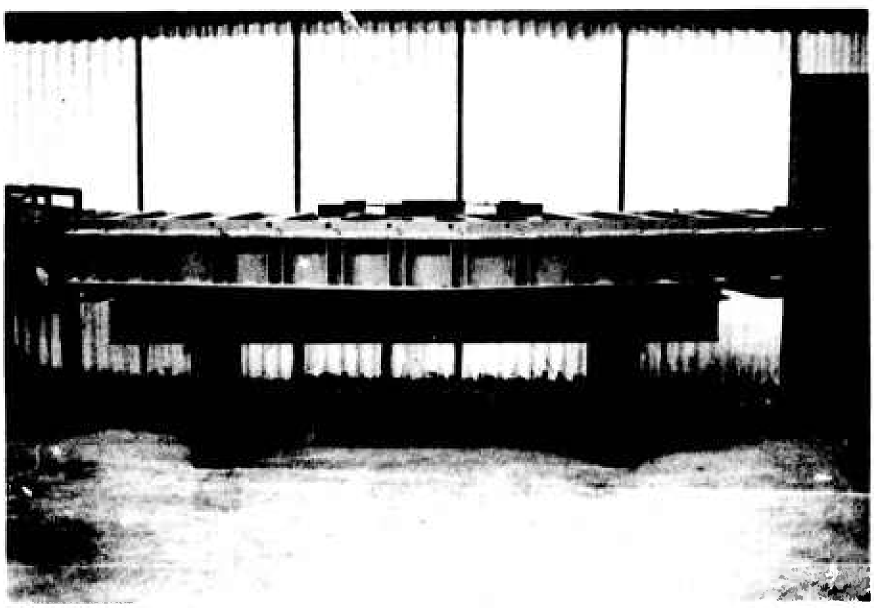
A model room 16-1/2 in. long, 11-7/8 in. wide and 8-1/2 in. high was constructed at the open end of the model. An approximately 15 percent opening (window) was placed in the center of the up-stream wall (toward



End View

Fig. B-1 Photograph of 1/12-Scale Model

End View



Side View

Fig. B-2 Photographs of 1/12 Scale Model the compression chamber) and an 18.8 percent opening (doorway) was placed at one side of the rear wall. These room and opening dimensions correspond to the full-scale dimensions of a test room constructed at the end of the Shock Tunnel and were used previously for blast-fire interaction tests. This sealed room is removable to allow for the installation of acetate diaphragms over the opening, and to provide the versatility for using different room and opening configurations.

Model Tests

For this initial feasibility study in the model, two quartz piezoelectric pressure gauges were installed in the room, one in the center of the logitudinal wall furthest from the door and the other in the center of the floor. The output of gauges went through charge amplifiers to a 14 channel FM tape recorder.

About 25 tests, with compression chamber pressures ranging from 2 to 15 psi, have been conducted. The data from some of these tests are presented in Fig. B-3, a plot of compression chamber pressure vs maximum pressure in the room. Each data point represents the average of the readings from the floor and wall gauges for a single test.

Sample pressure gauge traces from 2, 5, 10 and 15 psi compression chamber pressure tests are presented in Figs. B-4, B-5, B-6 and B-7.

The positive phase duration data from these same tests is presented in Fig. B-8. It was impossible to obtain accurate positive phase duration data from the pressure gauge traces because of indeterminate base line shift and the very small intersection angle of the base line with the pressure record. Therefore, the duration data presented in Fig. B-8 was obtained using a sensitive leaf switch located in the doorway in the back wall of the room. Sample data obtained with this leaf switch is presented in Fig. B-9.

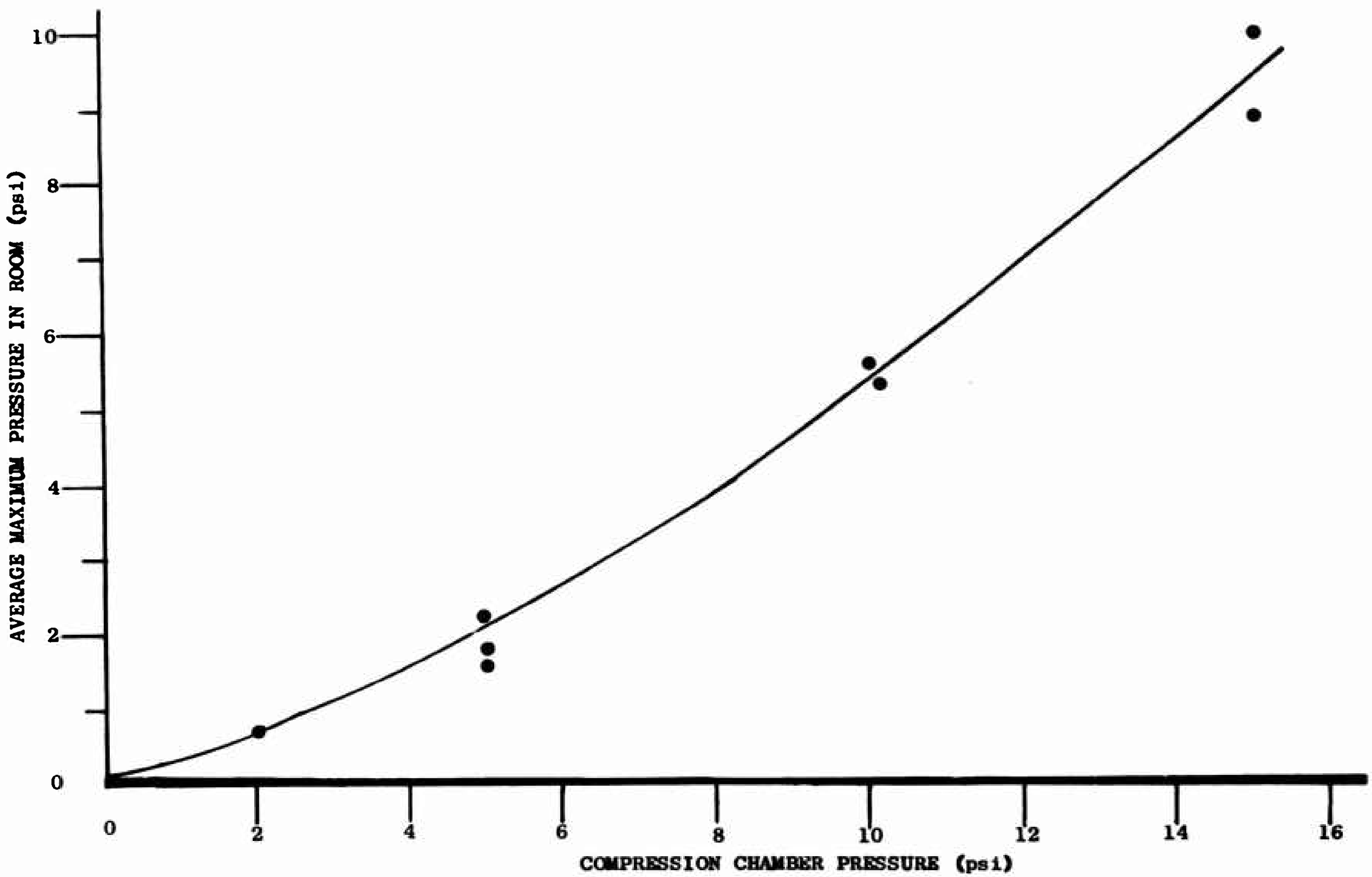


Fig. B-3. Plot of Compression Chamber Pressure vs Average Maximum Pressure in Room

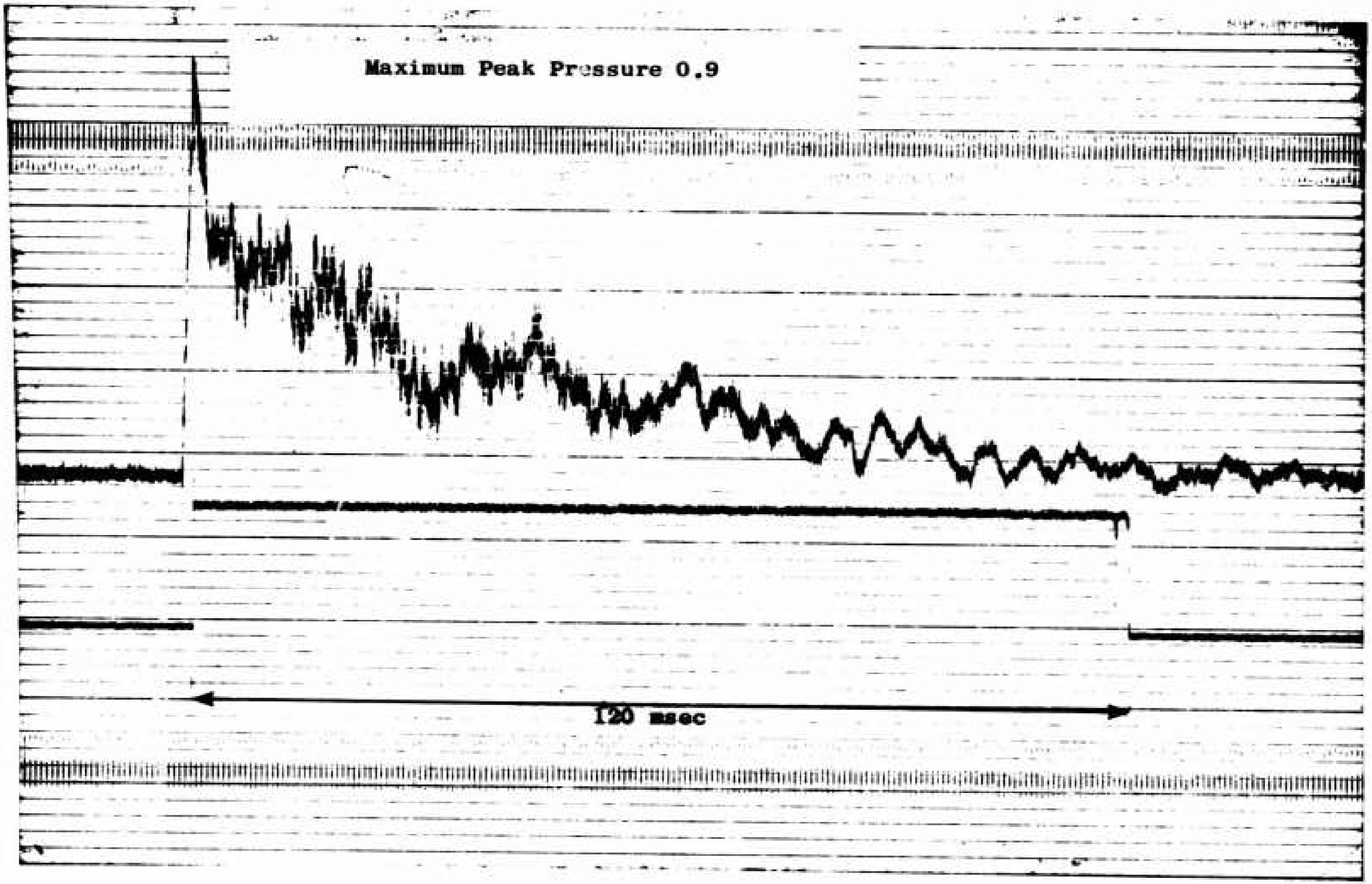


Fig. B-4. Pressure Gauge Traces, Chamber Pressure psi.

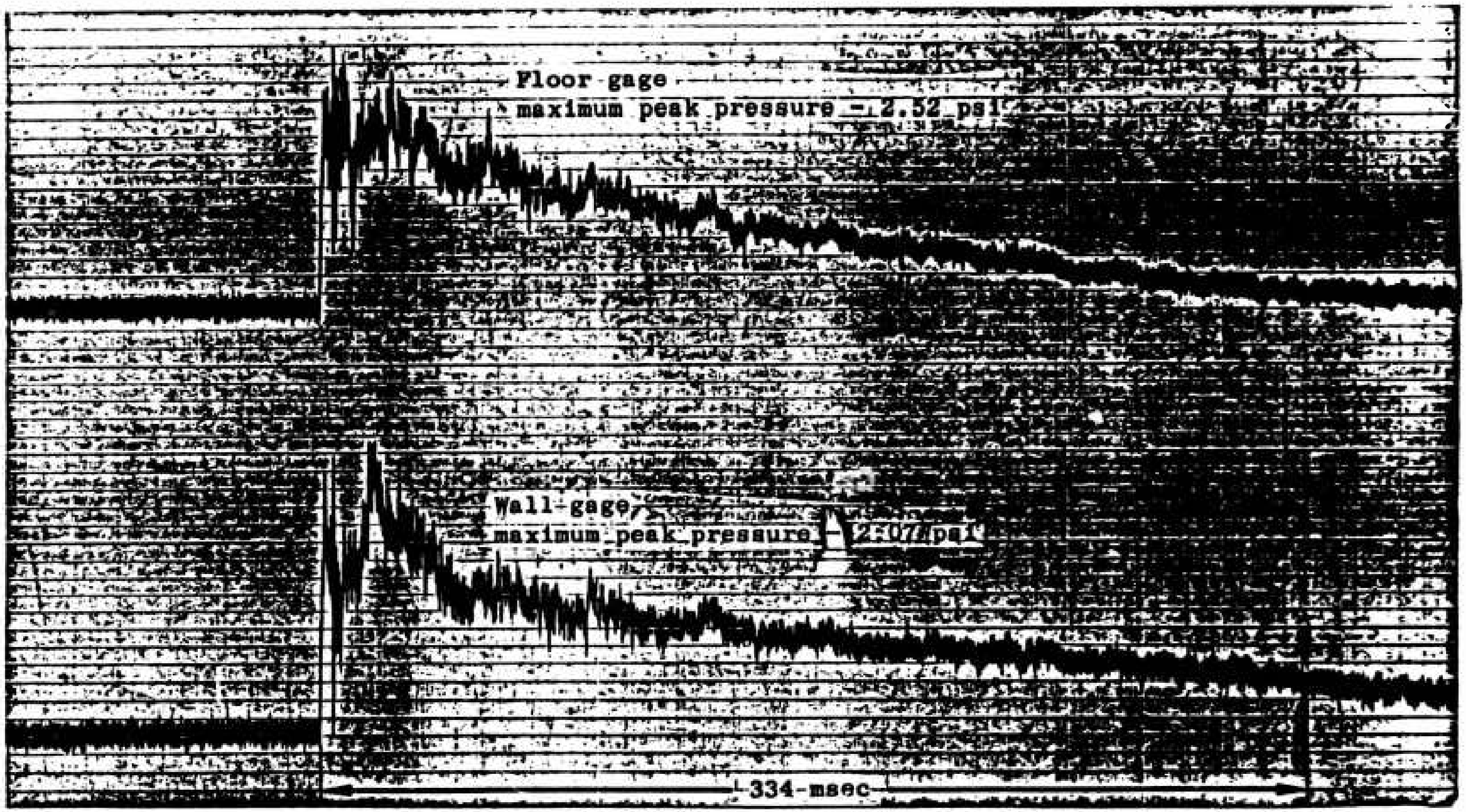


Fig. B-5. Pressure Gauge Traces for Test 09-27-72-01, Chamber Pressure 5 psi.

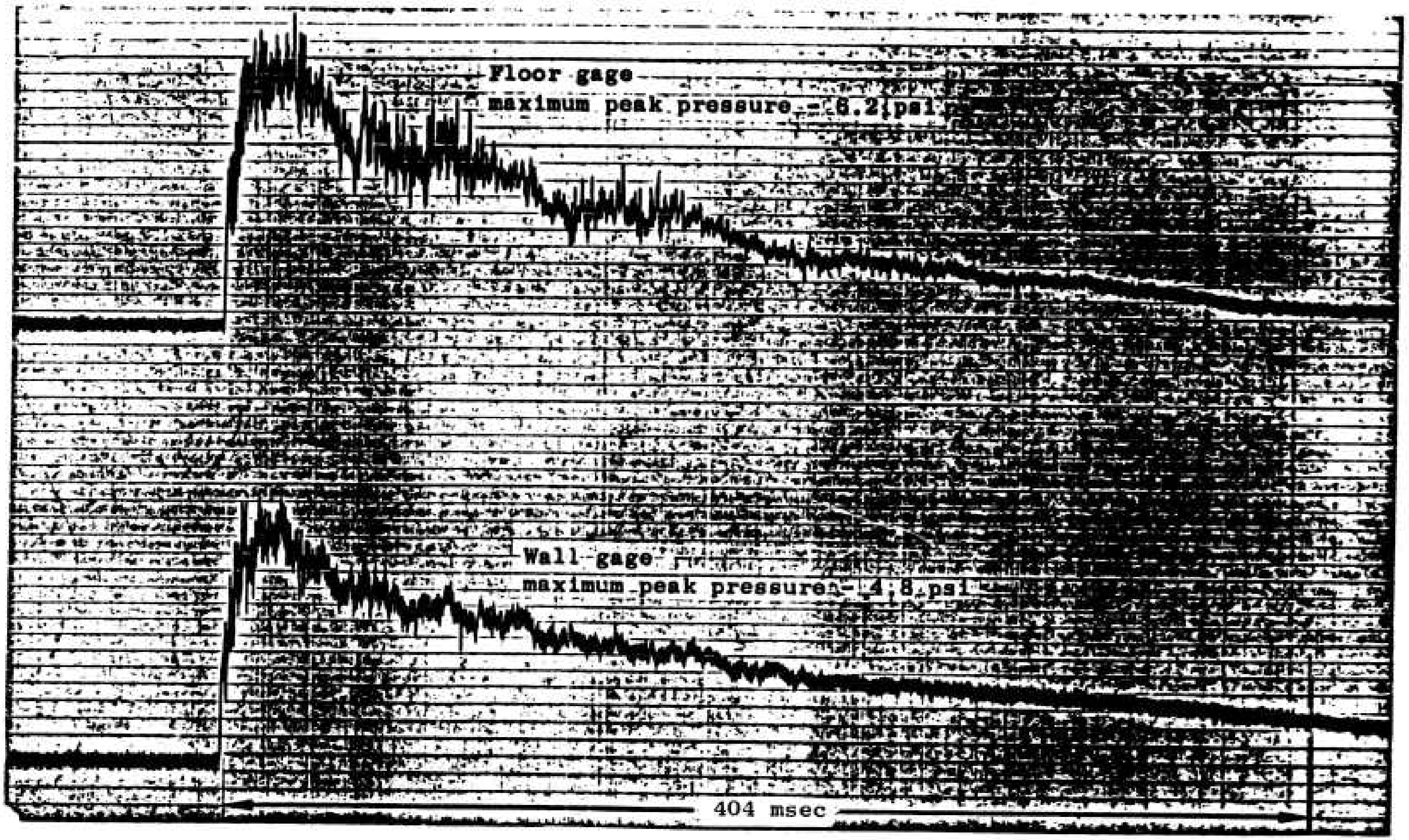


Fig. B-6. Pressure Gauge Traces for Test 09-26-72-01, Compression Chamber Pressure 10 psi

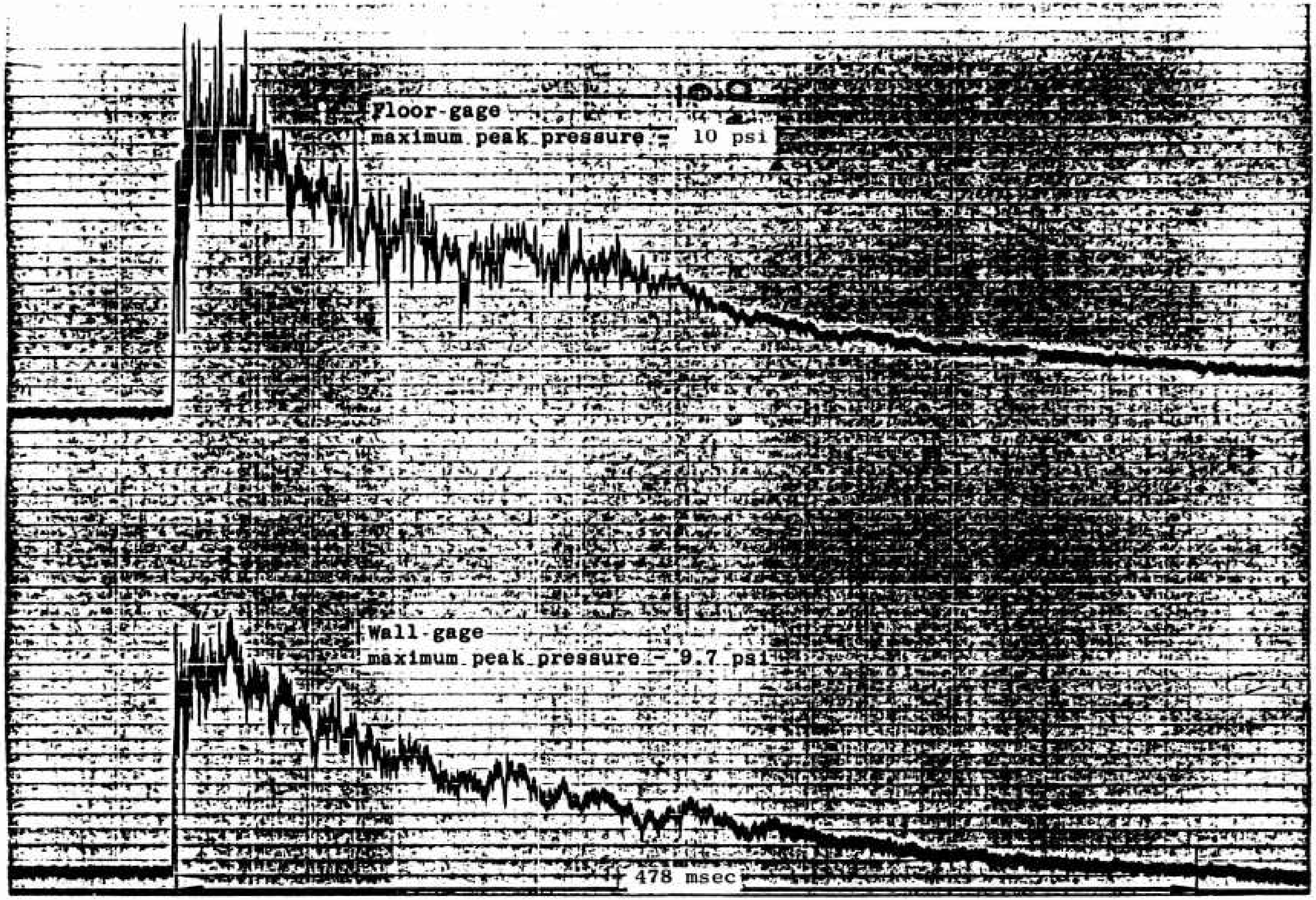


Fig. B-7. Pressure Gauge Traces for Test 09-27-72-02, Compression Chamber Pressure 15 psi.

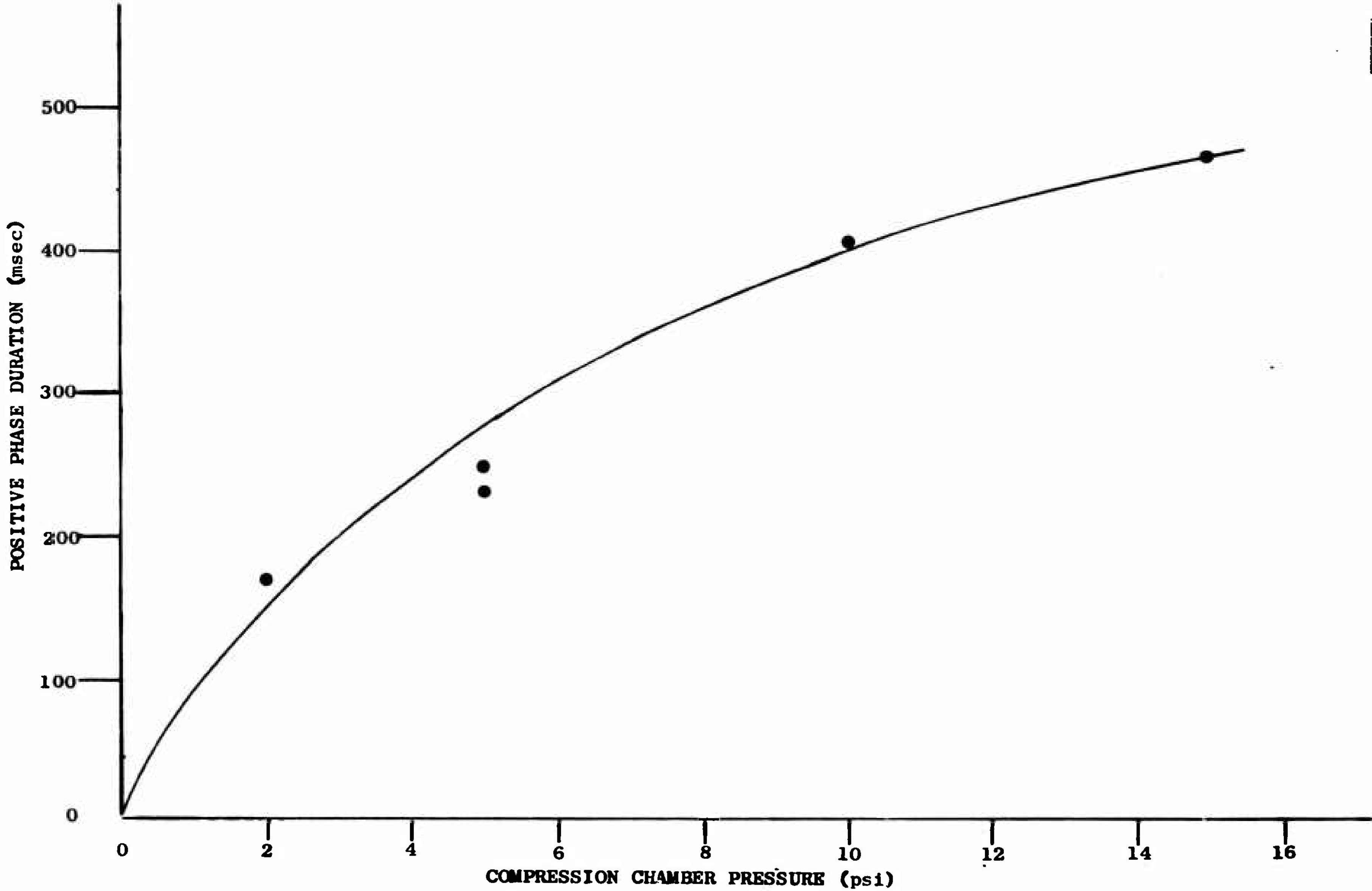


Fig. B-8. Plot of Compression Chamber vs Positive Phase Duration.

DESIGN CALCULATIONS

The following design/analysis computations were for the 1/12 scale model [(1 in. = 1 ft) of the proposed blowdown chamber at Fort Cronkite.]

Parameters

Scale 1 in. = 1 ft

Material A36 Steel

$F_y = 36{,}000$ psi

$F_{vy} = 20{,}000$ psi

Load $p = 10$ psi

or $p_u^* = 40$ psi

---

* We will assume a safety factor or yield of 4.0 (similar to boiler code).

Safety Factors

F.S. = 4.0

It is recommended that the model be designed for operation at 10 psi or less, but a proof testing should be performed at 20 psi then a safety valve set at 10 psi when installed.

Design Allowables (Ref. AISC Manual of Steel Construction)

Steel

$$\left.\begin{aligned} F_t &= 9{,}000 \text{ psi} \\ F_u &= 5{,}000 \text{ psi} \end{aligned}\right\} \text{for A36}$$

Welds

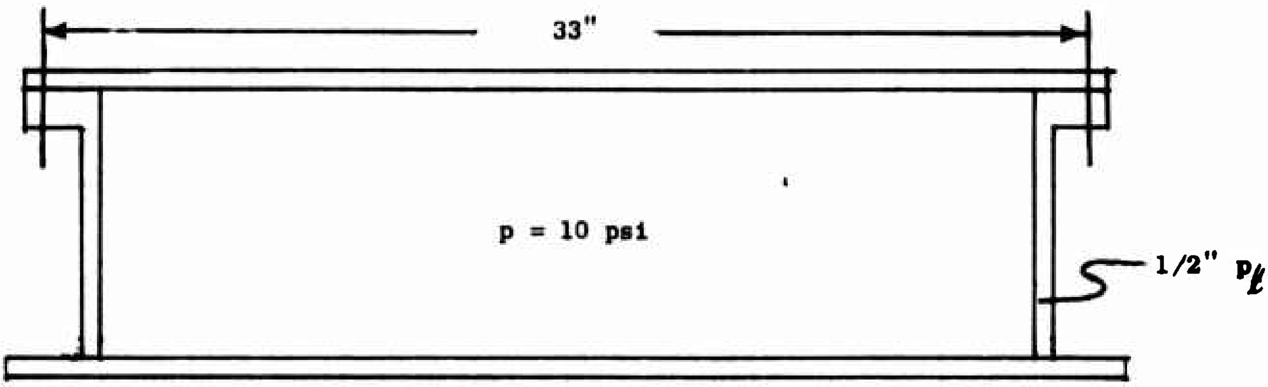
Use 4,000 psi for fillet welds or 1/4 is good for 700 lbs/in.

Bolts

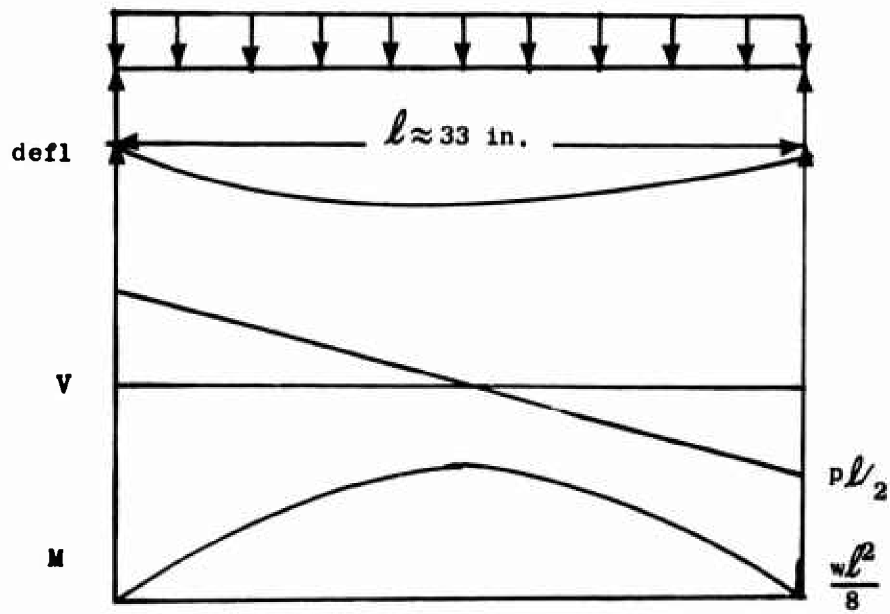
A325 bolts use 20,000 allowable

	Tension	Shear (Single)
5/8	6.13^k	2.3^k
3/4	8.88^k	3.1^k

First. A general look at the concept. With a bolted on cover the top will act like a one-way slab with a 30 in. span (approx.).



Top as a simple beam



$$M_{max} = \frac{10(33)^2}{8}$$

$$= 1360 \text{ in. lbs/in.}$$

$$V_{max} = 16.5(10)$$

$$= \underline{\underline{165 \text{ lbs/in.}}}$$

<u>Check Stress</u> (Flexural)

$$\sigma = \frac{6M}{bt^2}$$

$$= \frac{6(1360)}{1 \times (\frac{1}{2})^2}$$

$$\sigma = 32{,}700 \text{ psi} > 9{,}000 \text{ psi}$$

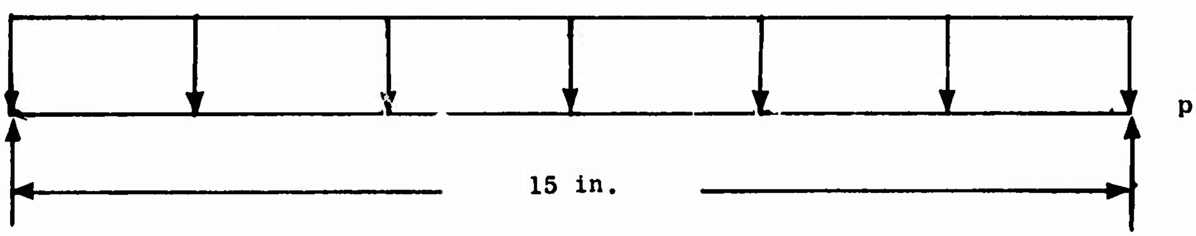
<u>Deflection</u>

$$\delta = \frac{5\, p\ell^4}{384EI}$$

$$= \frac{5(10)(33)^2(33)^2\, 12}{384 \times 30 \times 10^6 (\frac{1}{2})^4}$$

$$\delta = \underline{0.495 \text{ in.}}$$

Before redesigning lets look at the 12 in. span (+ 3 in. for bolt line)



$$\sigma = \frac{6M}{bt^2}$$

$$= \frac{24(10)(15)^2}{8}$$

$$\sigma = 6750 \text{ psi} < 9{,}000 = \text{safe}$$

$$\delta_{15} = \delta_{33}\left(\frac{15}{33}\right)^4$$

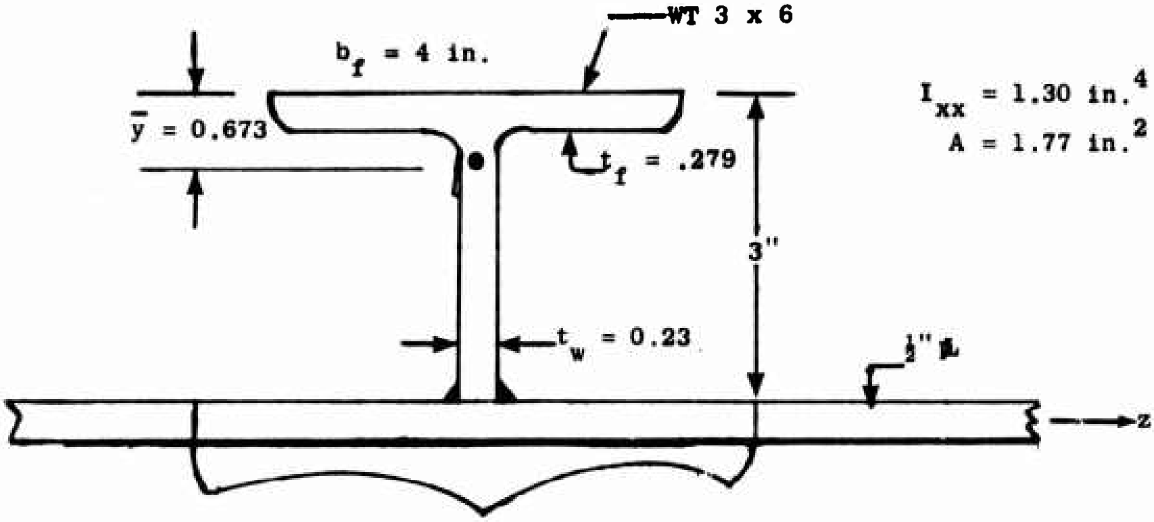
$$= \frac{0.495}{(2.2)^4}$$

$$\delta = 0.021 \text{ in.} < \frac{\ell}{360} = 0.042$$

Hence, the 12 in. portion is safe at t = ½ in. without stiffeners, etc.

Design Stiffeners for 30 in. section

Assume stiffeners 12 in. center to center



let b = 20_t = 10 in.

Find I_{zz} and c.g.

$$I_{zz} = 1.30 + 1.77\,(2.327)^2 + 1/3\ 10(\tfrac{1}{2})^3$$

$$= 1.30 + 9.61 + .42$$

$$I_{zz} = 11.33 \text{ in.}^4$$

$$A = 6.77 \text{ in.}^2$$

$$6.77\,\bar{z} = 1.77(2.33) - 5(0.25)$$
$$= 4.12 - 1.25$$
$$= 2.87$$
$$\bar{z} = 0.424 \text{ in.}$$

$$I_{oo} = I_{zz} - Ad^2$$
$$I_{oo} = 11.33 - 6.77(0.424)^2$$
$$\underline{\underline{I_{oo} = 10.12 \text{ in.}^4}}$$

$$C = 3 - 0.424$$
$$= 2.576$$
$$\underline{\underline{S = 3.93 \text{ in.}^3}}$$

Check Stress and Deflection

$$M = \frac{12(10)(33)^2}{8}$$
$$= 16{,}320 \text{ in. lbs/Stiffener}$$

$$\delta = \frac{M}{S} = \frac{16{,}320}{3.93}$$

$$\delta = 4160 \text{ psi} < 9000 \therefore \text{safe}$$

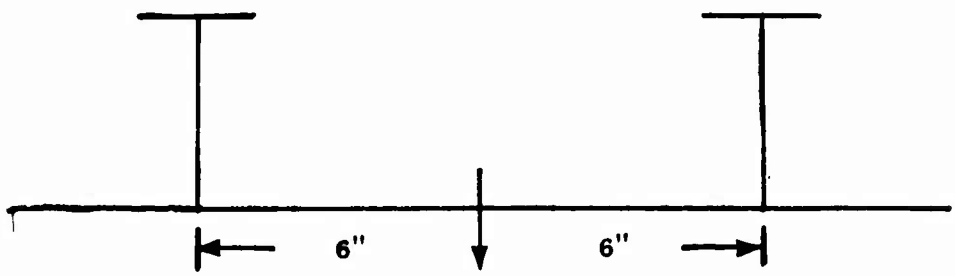
$$= \frac{5(120)(33)^2(33)^2}{384 \times 30 \times 10^6 \times 10.12}$$

$$\delta = 0.056 \text{ in.} < \frac{\ell}{360} = 0.092 \text{ in.}$$

this is safe

Holddown Bolts

Assume one bolt between each "Tee" stiffener



T_{eu} = 10 psi x 12 in. x 16.5 in.

5/8 - A325 bolt safe, but plate needs stiffening.

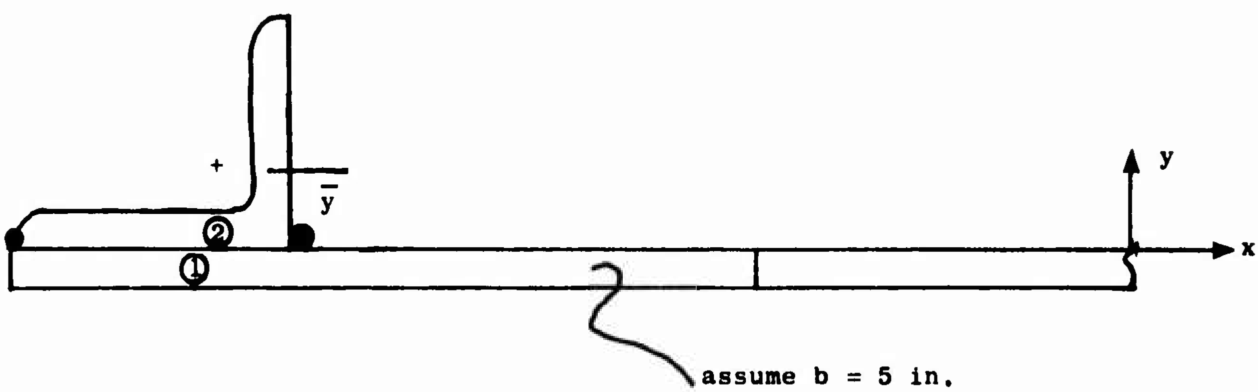
Try an edge 2-½ x 2-½ x 3/8

$I = 1.23\ 0.984\ \text{in.}^4$

$S = 0.724\ 0.56C\ \text{in.}^3$

$A = 1.73\ \text{in.}^2$

$\bar{y} = 0.762$ in.



Item	A	$\bar{y}$	$A\bar{y}$	$A\bar{y}^2$	I_o
①	2.5 in.^{2}	- 0.250	- 0.625	0.156	0.052
②	1.73 in.^{2}	0.762	1.318	1.003	0.984
	4.23 in.^{2}		.693		

$\bar{y} = 0.164$ $\qquad I_{zz} - 2.195$

$$I_{oo} = 2.195 - 5(0.164)^2 = \underline{\underline{2.06 \text{ in.}^4}}$$

$$C = 2.5 - 0.16 = 2.34 \text{ in.}$$

$$S = 0.883$$

Stress

$$\frac{PL}{8} < M < \frac{PL}{4} = \frac{1975(12)}{4}$$

$$M < 5925 \text{ in./lbs}$$

$$\therefore \sigma < \frac{M}{S}$$

$$\sigma < \frac{5925}{.883} \text{ psi}$$

$$\sigma < 7000 \text{ psi} \therefore \text{safe}$$

Deflection

$$\frac{PL^3}{192EI} < \delta < \frac{PL^3}{48EI}$$

$$\delta < \frac{1975(1728)}{48 \times 30 \times 2.06}$$

$$\delta < .00115 \text{ in.} < \frac{\ell}{360} = 0.033 \text{ in.}$$

Check Welds

at the "T" line

$$V_{max} \simeq 1{,}000 \text{ lbs}$$

$$Q = 5 \text{ in.}^2 (0.424 + 0.25)$$

$$= 3.37 \text{ in.}^3$$

$$I = 10.12 \text{ in.}^4$$

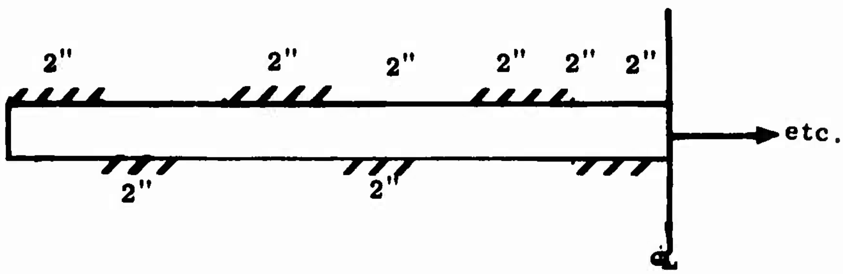
$$q = \frac{VQ}{I} = \frac{1975(3.37)}{10.12}$$

$$q = 660 \text{ lb/in.}$$

a 1/4 ◣ weld worth 700#/in.

∴ Use 50% of length as 1/4 welds staggered (solid)
then 50% of length half welded

i.e.



at the ℄

$V = 987.5$ lbs

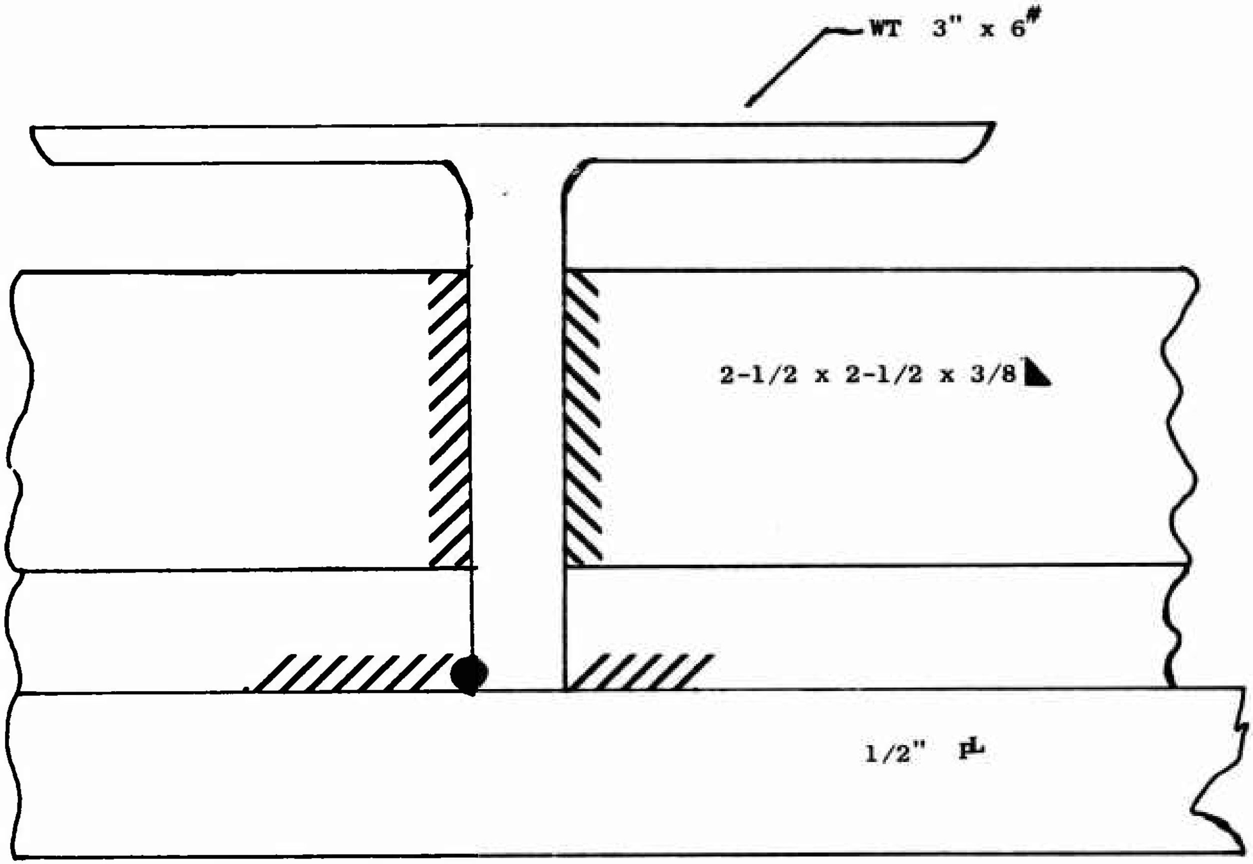
$I = 2.06$ in.^{4}

$Q = 2.5 \text{ in.}^2 (.25 + .164) = 1.03$

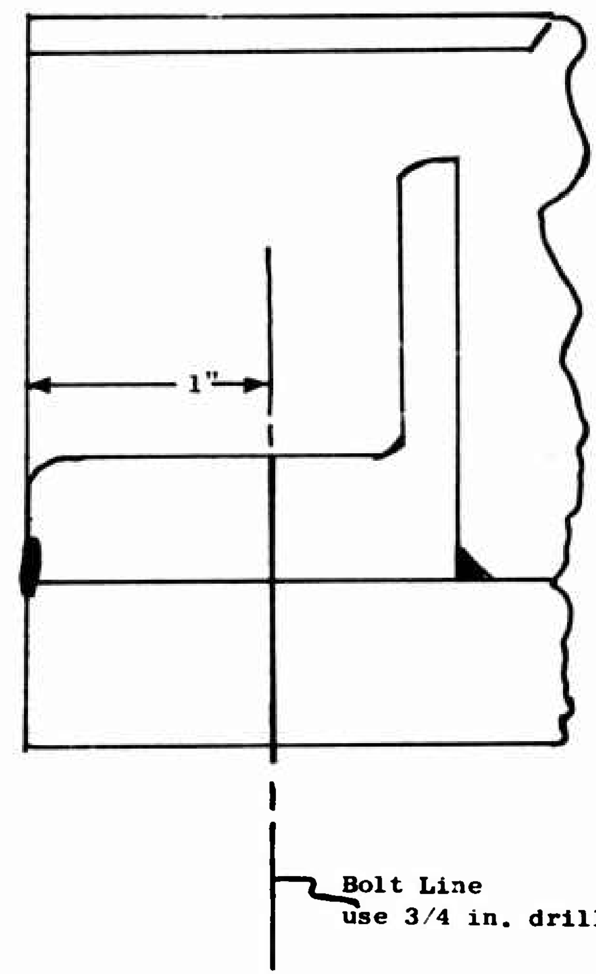
$q = \frac{VQ}{I} = 987.5(\frac{1}{2})$

≈ 500 lbs/in.

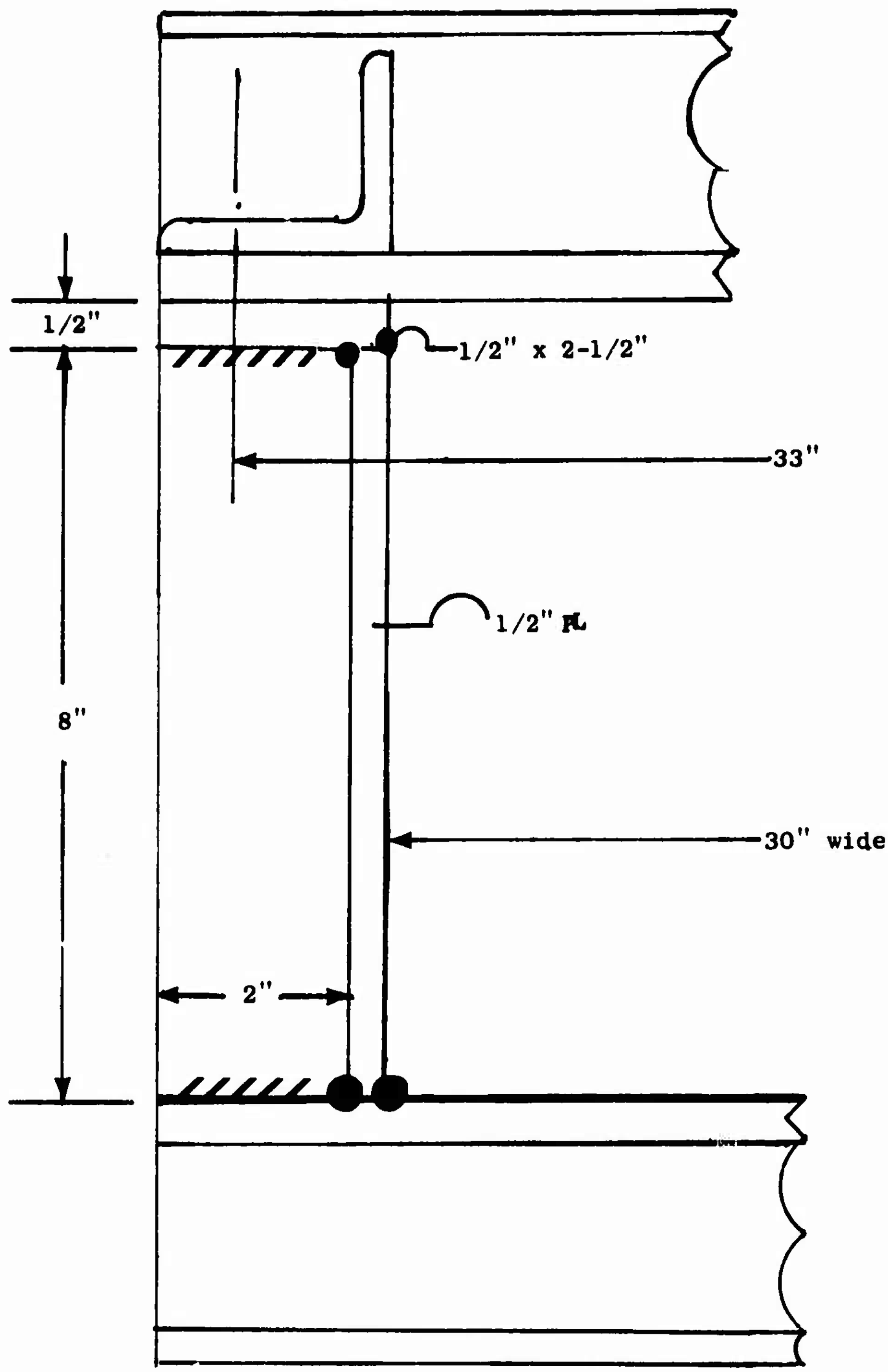
use same scheme as above

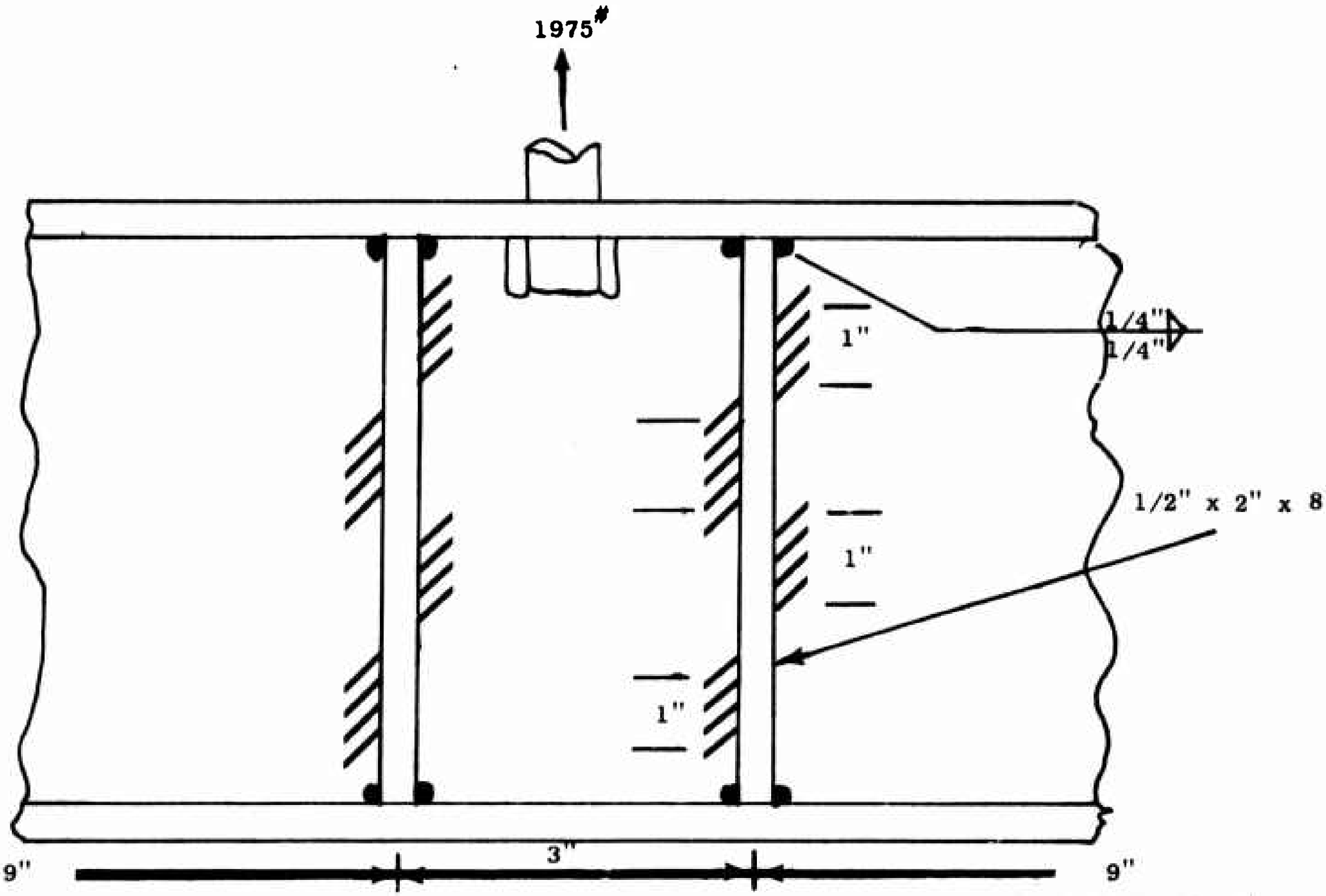


Build 30 in. portion like this on the top(omit 4 on bottom)



Check the side





<u>Check Top Plate at Bolt</u>

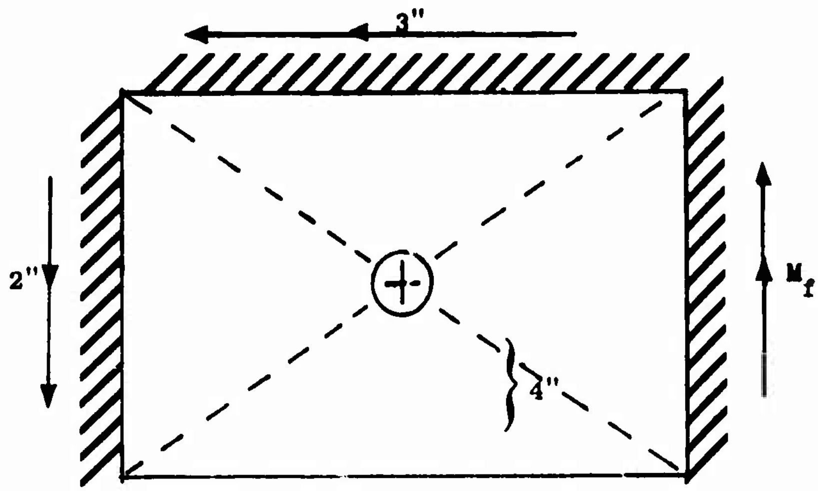
$$t = 1/2 \text{ in.}, \quad b = 2\text{-}1/2 - 3/4 - 1\text{-}3/4 \text{ in.}$$

$$M = \frac{P\ell}{4} = \frac{1975^{(3)}}{4'} \text{ in./lbs}$$

$$\sigma = \frac{6M}{bt^2} = \frac{6(1975)(3/4)}{1.75}$$

$$\delta = \underline{\underline{20,300 \text{ psi}}}$$

The foregoing is conservative as the area is actually a plate supported on three edges.



look at like yield line

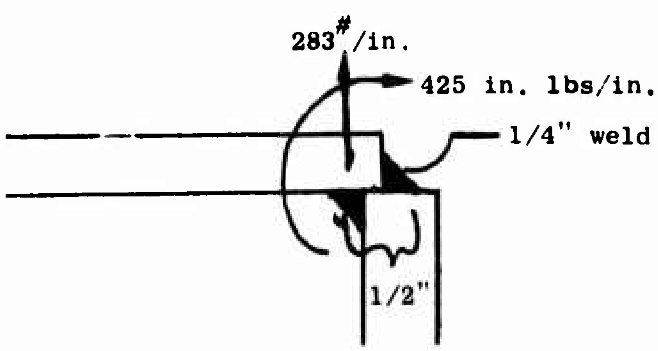
$$\frac{1975 \text{ lbs}}{7 \text{ in.}} \times 1.5 \text{ in.} = M$$

$$M = 425 \text{ in. lbs/in.}$$

$$\sigma = \frac{6M}{bt^2}$$

$$\sigma = 24(425)$$

$\sigma = 10{,}200$ psi in the plate



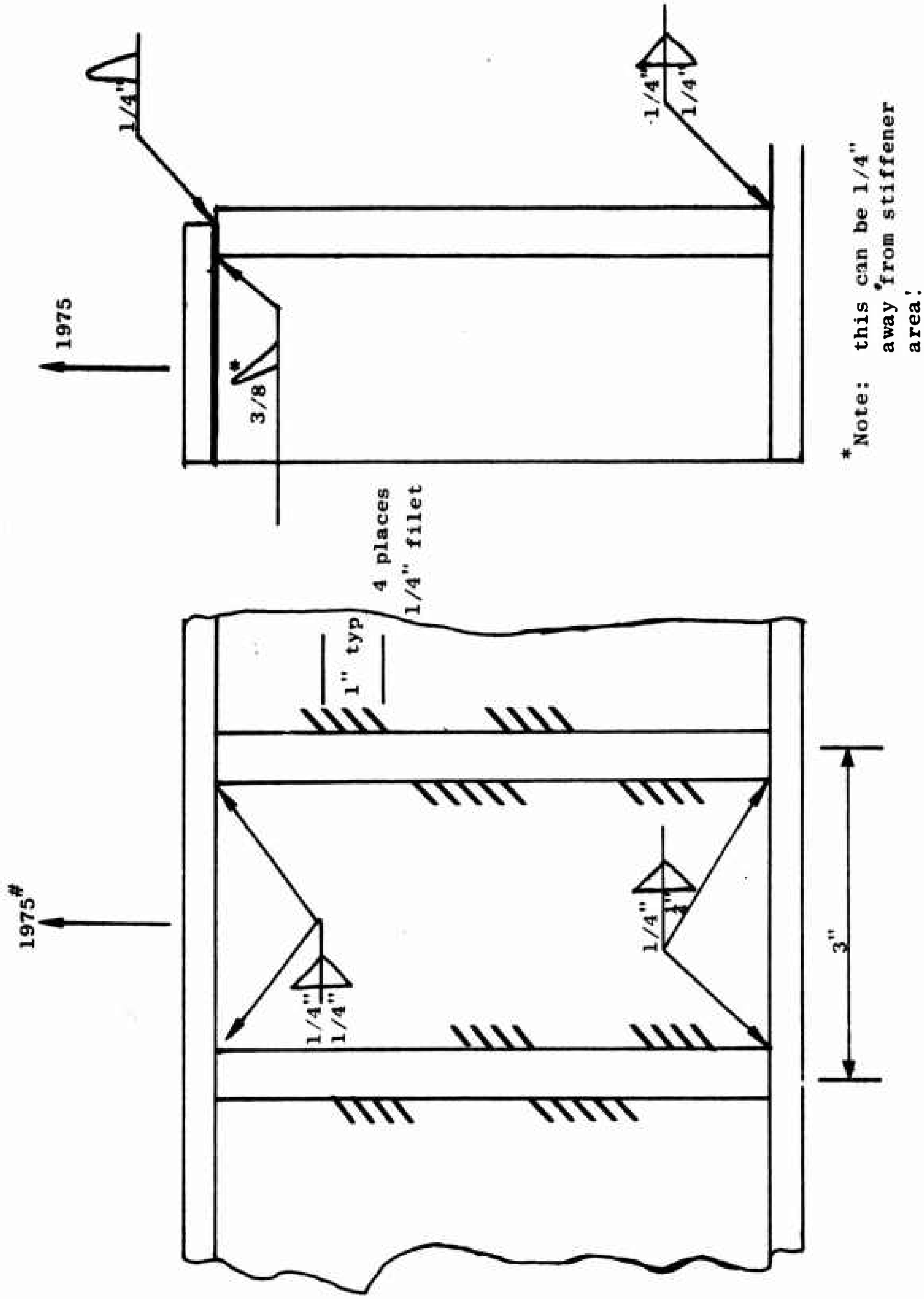
Look at Weld

weld load = 283 + (425) ÷ 1/2

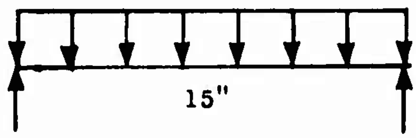
= 1133#/in.

∴ use 3/8 weld good for 1050

Final Stiffener Design

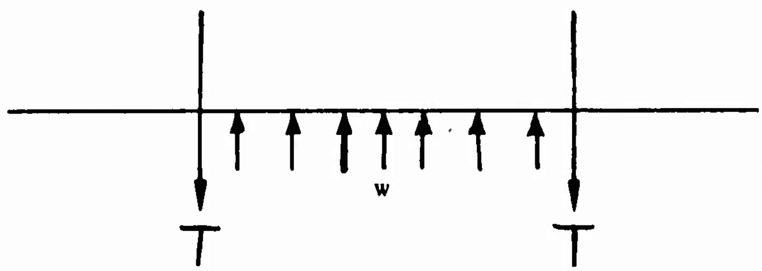


12 in. Region of Model



V = 75 lbs/ in.

Assume angle stiffener along edge as in 30 in. section of model



w = 75#/in.

$$I = 2.06 \text{ in.}^4$$

$$S = 0.883 \text{ in.}^3$$

$$M = \frac{w\ell^2}{10}$$

$$M = \sigma S = 9{,}000\,(0.883)$$

$$= 7{,}950 \text{ in./lbs max}$$

$$\frac{75(\ell^2)}{10} = 7{,}950$$

$$\ell \leq 32.5 \text{ in.}$$

$$\delta = \frac{5w\ell^4}{384EI} \leq \frac{\ell}{360}$$

$$\therefore \quad \ell^3 = \frac{384EI}{360 \times 5xw}$$

$$\ell \leq \left(\frac{1.065 \times 30 \times 2.06}{5 \times 75}^{1/3}\right) x\ 10^2$$

$$\underline{\ell \leq 56 \text{ in}}/\delta \approx 1/4 \text{ too much}$$

Space 5/8 bolts @ 24 in.

$P = 1{,}800$ lbs $\therefore$ safe

$M = 4{,}320$ in./lbs

$$\sigma = \frac{4{,}320}{0.883}$$

$$\underline{\underline{\sigma = 4{,}880 \text{ psi}}} \therefore \text{ safe}$$

$$\delta \leq \frac{5(75)(24)^2(24)^2}{384 \times 30 \times 10^6 \times 2.06}$$

$$\delta \leq \underline{\underline{0.0052}} \text{ in. safe}$$

$$V = 12(75)$$

$$\underline{\underline{V = 900 \text{ lbs}}}$$

$$q = \frac{VQ}{I}$$

$q \simeq 1/2, V = 450^{\#}$/in. (ref. p. C-17)

Use welding scheme shown on p. C-17.